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DIVISION 500 – STRUCTURES

SECTION 501 - TEMPORARY BRIDGE

501.1 - GENERAL

The contract administrator inspects the construction, maintenance, and removal of a temporary bridge and helps coordinate the activities of various personnel in providing traffic safety.

501.3 - CONSTRUCTION OPERATIONS

A. Approval of Plans:

The contractor will provide the contract administrator with detailed plans designed and stamped by a licensed professional engineer for the temporary bridge item. The contract administrator will forward plans for temporary bridge structures to the Construction Bureau office for distribution and approval.

B. Inspection:

The contract administrator must inspect the work to ensure that the contractor builds the bridge in accordance with the submitted plans. Particular attention should be focused upon preparation of the abutment foundations and obtaining a thoroughly compacted foundation. The contractor is responsible for the maintenance of the temporary bridge throughout its use. Payment of this item should consist of partial estimates for the three stages (construction, maintenance, and removal), rather than 100% upon erection.

C. Opening Bridge to Traffic:

1. <u>Signs and Striping:</u> Signing for detours onto the temporary bridge will be provided and maintained by the contractor under Item 619.1 Maintenance of Traffic. Since each temporary bridge location and its approaches are individually designed, check the plans and special provisions for lighting, special barricades, flashing arrows, etc., which might be required in addition to normal signing. If the approaches are to be paved, striping should be completed and existing lines on the approaches removed as necessary to ensure that the traveling public does not get confused.
2. <u>Notification:</u> The contract administrator should notify the NHDOT public information officer, the NHDOT wide load permitting office, the construction office, and local authorities at least one week prior to opening a temporary bridge and again prior to changing traffic to the new alignment. Phone numbers for the DOT offices may be found in section 900 of this manual.

SECTION 502 - REMOVAL OF EXISTING BRIDGE STRUCTURES

502.1 - GENERAL

This item consists of the removal and satisfactory disposal of the existing bridge structure. Particular care should be taken by the contract administrator to determine the exact limits of removal as denoted on the plans prior to commencing the operation. Considerable misunderstanding can result when limit lines are not clearly designated on the plans. A common example of this situation is in determining whether a wing wall is considered a portion of the abutment structure or a separate retaining wall structure. Clarification of removal limits at an early date will eliminate considerable confusion.

502.3 - CONSTRUCTION OPERATIONS

The contractor shall dismantle the existing structure in a manner that will not cause damage to persons or property nor interfere with the traveling public. A removal plan shall be submitted and accepted before work can take place.

Prior to removal operations by the contractor, the contract administrator can expedite the removal operation by ensuring that all utilities suspected of being in the vicinity of the structure have been identified, relocated, or terminated. This should be discussed at the pre-construction conference.

In the event that the contractor chooses to use explosives in the removal operation, special consideration must be taken to ensure that there will be no damage to the new work. If the contract administrator is in doubt as to the effects of the blasting, the District Construction Engineer should be contacted for a further study of the operation.

For removal operations adjacent to waterways, special attention should be paid to applicable regulations of the NH Division of Water Supply and Pollution Control as detailed in Section 107 of the Standard Specifications.

Contractors must follow extra stringent safety procedures when removing structures with lead paint. Make sure that special consideration has been given in these cases.

SECTION 503 - COFFERDAMS

503.1 - GENERAL

A cofferdam is a form of shoring, generally of a temporary nature, that is constructed for the purpose of keeping water and earth out of a structure excavation. The type of cofferdam usually depends on the water conditions encountered. Small streams can usually be handled with an earth embankment or diversion channel. Deeper rivers will require cribs, sheeting, or caissons to hold back water and earth pressures and provide a safe work area on or below the river bottom. Cofferdams shall be as watertight as necessary to permit de-watering by pumping and shall provide a reasonably dry work area for concrete masonry operations.

A plan must be submitted for documentation prior to the start of work. It is the contract administrator's duty to ensure conformance with this plan. If changes are made in the cofferdam installation, an as-built set of plans, stamped by a New Hampshire P.E. shall be submitted.

503.3 - CONSTRUCTION OPERATIONS

A. Temporary Diversion Channel:

1. Size and Location: The size of the diversionary channel should be sufficient to carry peak seasonal flow. Channels should be located where economically feasible, taking precautions to minimize permanent damage to nearby trees, vegetation and wildlife.
2. Water Pollution and Erosion: The contract administrator and contractor must become familiar with all instructions concerning water pollution and erosion contained in the contract documents. The contractor should be made aware of practices to minimize pollution, such as excavating channels in the dry, constructing stone lining or other erosion protective measures, and allowing sufficient time for settlement prior to diverting the stream.

B. Embankment Cofferdams:

1. Puddle" Cofferdams: For excavations less than 10 ft (3 m) deep in stable soils, a satisfactory work area often can be provided by using the excavated material in the construction of a dike.

2. Embankment Cofferdams: For deeper excavations, a more sophisticated construction technique is required. The size of the embankment depends on the distance between depth of excavation and anticipated high water level subject to the angle of repose of the fill material. It is advisable to protect the embankment against current erosion and subsequent water pollution by placing ledge and boulders around the perimeter. Usually, stone slope protection can be placed concurrently with the construction of the earth embankment. Also, at times, soil conditions necessitate the construction of an impermeable core in the embankment.

C. Sheetimg Cofferdams:

Sheeting Cofferdams: Sheeting is often used in cofferdam construction in deep water and heavy current conditions; sign structure foundations; and adjacent to existing highway structures and railroad tracks. Construction usually begins by making a frame that, apart from being a primary structural member, acts as a template that holds the sheets to plan dimension during driving. The contract administrator should strongly recommend to the contractor to make the initial alignment of the frame as accurate as possible so that any minor movement of the sheets during driving will be of little consequence. The frame must be aligned when it is horizontal and the sheets should be driven plumb for the best results. Should the sheeting travel or lean excessively it may be necessary to extract the sheets and re-drive or increase the size of the cofferdam.

Care should be taken to drive the sheets well below the bottom elevation of the excavation (toe-in). The contract administrator can put a grade on the frame to be used as a reference in determining the tip elevation of the sheets. These elevations should be checked before computation of the tip elevation and they should never be used as a bench mark for future work. Where the cofferdam is to be the form for a seal pour, additional care should be taken with the alignment of the frame and the driving of the sheets to ensure that the minimum concrete dimensions (inside the projections of channel and Z type sheets) as shown on the plans are maintained. An adequate sump must be provided outside of the forms and inside the cofferdam to allow pumping with minimal seepage through the forms.

Timber Sheet Pile Driving Characteristics: A single or double row of timber sheet piling is sometimes used if only an earth bank is to be supported. Where water tightness is desired, or if earth pressures are large, some form of tongue and groove sheeting is preferable. Triple sheeting withstands driving better than single planks because defects cannot extend through the entire sheeting, and warping is minimized.

In placing timber sheeting, the tongue should always lead. That is, the groove of the timber being driven should be sliding down over the tongue of the previously driven timber in order to prevent clogging in the groove.

Steel Sheet Pile Driving Characteristics: Interlocking steel sheet piling may be driven by one of two basic methods.

Method #1 - Driving individual piles: A single pile or pair of piles is driven at one time. The leads should be vertical and stable with the hammer centered over the neutral axis of the pile. Driving piles in pairs generally makes guiding easier. This method is particularly advantageous when a stable and level foundation can be provided for the pile driving equipment. The best practice in driving is for the ball end to lead. This prevents soil from becoming trapped in the interlock.

Method #2 - Continuously driving a preassembled panel of piles: Piling is assembled in wall form first, and driven continuously along the line. It is necessary to be able to set the piling with both axes plumb and hold the hammer rigid. If the contractor uses swinging leads, the leads and hammer should be held rigidly in the same vertical plane. Vibration in the hammer or the pile will result in the piles being driven out of alignment. It is advisable to drive Z piles in pairs.

Use a guide form or guide walers to get well-driven, aligned sheeting. Sometimes the walers can be built in a movable trestle-like form. Distance between walers should be slightly wider than the back-to-back distance of the sheeting. To provide guiding stability for installing successive sheets, a wood wedge can be placed in the trough of the previously driven sheet, as illustrated in the figure below.

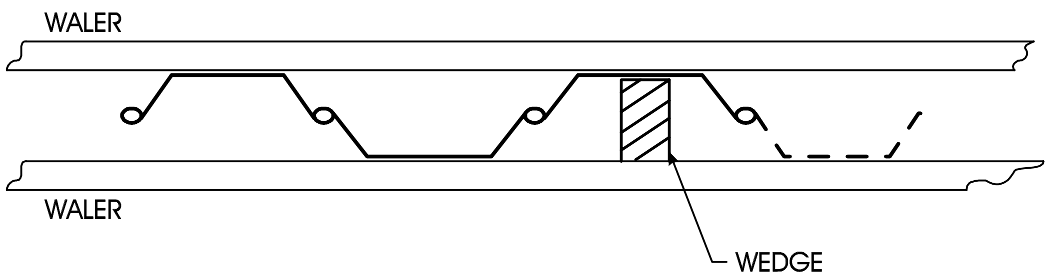


Figure 5-1: Guiding Sheet Piles with Wedge

D. Obstructions:

If borings or other information show obstructions, drive sheet piling in panels. When an obstacle is hit, stop driving and move the hammer to the next pile that can be driven. With piles on both sides of the obstacle acting as guides, it often is possible to drive through the obstacle. Increasing the number of hammer blows helps.

E. Driving Tips:

1. Plug the open interlock. Material is prevented from clogging the leading interlock in the first pile by forcing a bolt or similar object into the open case.
2. Drive short sheet piles in soft ground to full depth singly or in pairs.
3. To prevent creep in driving long piles or in hard ground, proceed as follows:
   - Set guide walers along the line of sheeting.
   - Drive a pair of sheet piles to partial depth.
   - Set a panel of a dozen single piles or pairs in the walers.
   - Drive the last pile or pair in the panel part way.
   - Drive the piles between the first and last piles or pairs to full depth.

- Drive the first pile to full depth.
- Drive the last pile two-thirds its full depth to act as a guide for the first pile of the next panel.

A good rule of thumb recommends that no sheet pile be driven more than one-third its length before adjacent sheet piling is driven.

4. Do not over-drive. Sometimes a driven pile may be drawn down by the next one being driven if the ground is very soft, or where high frictional forces develop in the interlock. To counteract this draw down, bolt the piles to a stiff wale, and if it happens before this precaution is taken, it usually is better to lengthen the drawn pile than to try to jack or lift it.

F. Cofferdam Uplift

After the cofferdam has been constructed, excavation completed, required piles driven, and soil reaction observed; calculate the hydrostatic uplift for the anticipated water level during the critical period that the cofferdam is to be dewatered. Since these values are assumed in the design, it is important that the actual dimensions are checked in the field during construction. Any variations should be brought to the attention of the Bridge Design Engineer and/or District Construction Engineer.

The example and graphs found at the end of this section are given as a rule of thumb for dewatering a cofferdam after the foundation seal has been poured. Using the minimum depth of the concrete seal, the graph can be used as a quick reference guide for determining the stability of the foundation seal. The height of water should be measured from the vent elevation of the cofferdam. Calculations should be performed if the variables plotted approach the dividing line.

When a foundation seal is to be poured on rock, the bottom of the cofferdam is to be inspected for total excavation and condition of the rock surface. This inspection is performed by divers under State contract. Contact your District Construction Engineer with sufficient lead time to arrange for this inspection when needed.

After the foundation seal has been poured on rock, an independent coring crew will be brought in to take core samples through the seal and at least one foot into the rock at each corner and one in the middle. These cores are inspected for voids, seams, or unsatisfactory concrete. These cores shall be labeled and placed in core boxes and remain on the project for future reference.

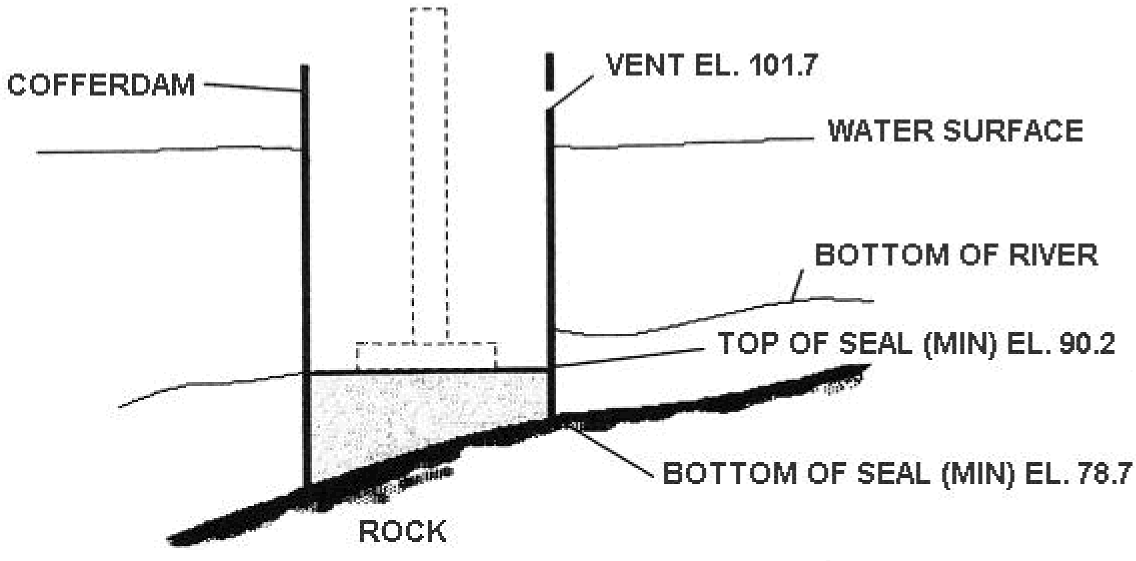


Figure 5-2: Cofferdam uplift

EXAMPLE
Seal height of 11.5 feet - from the following graph the maximum head of water equals 14.4 feet.
Actual head of water = 101.7 – 90.2 = 11.5 feet.
Therefore, the seal will be stable.
If the vent elevation were 106.6 instead of 101.7, the seal would be in danger of uplift.

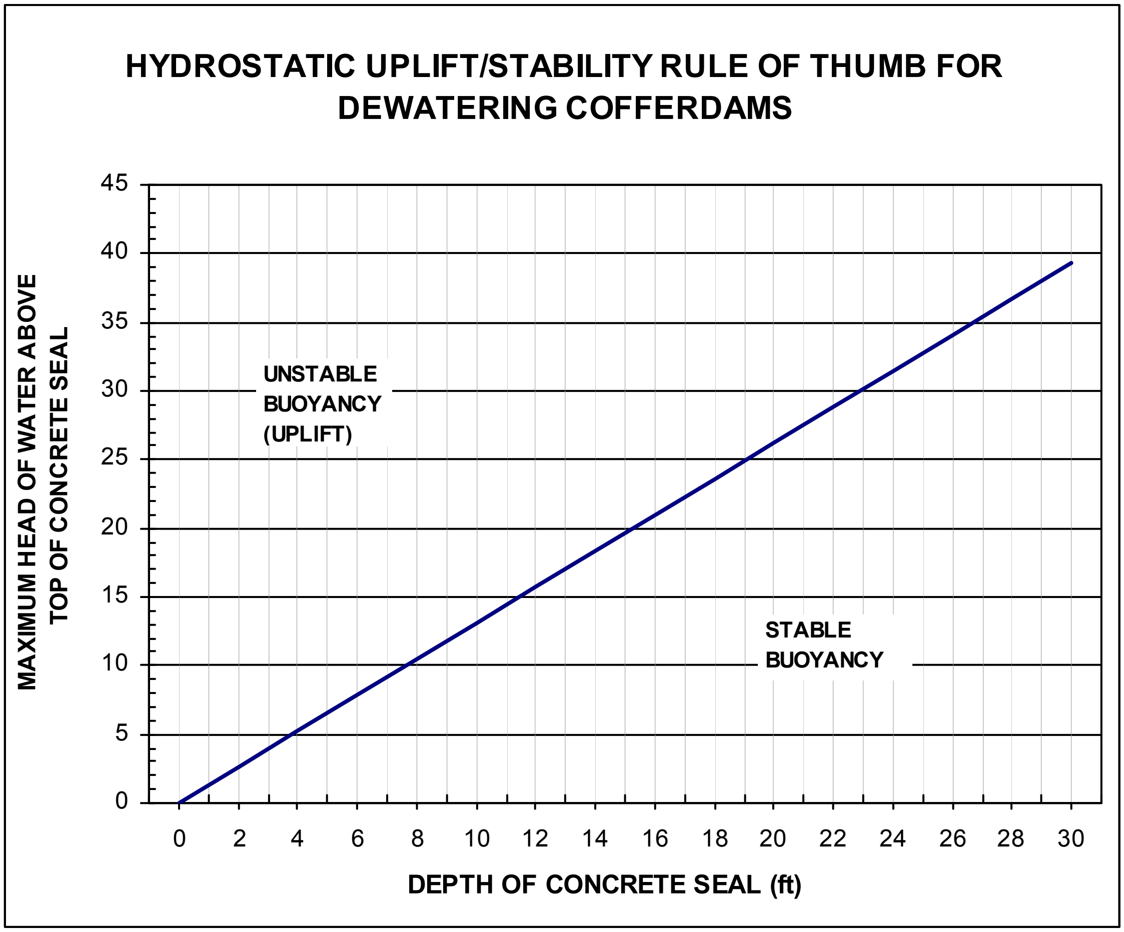


Figure 5-3(E): Cofferdam Uplift Graph (English)

<u>Figure 5-3(E) Assumed Values:</u>
Unit Weight of Concrete 144.2 lbs/cubic foot
Unit Weight of Water 62.43 lbs/cubic foot
The weight of the sheeting and the framing and the friction of the seal against the sheeting are not considered

Figure 5-3(E) balances the hydrostatic uplift pressure of the water with the weight of the concrete and any case that is close to the unstable buoyancy side of the line should be brought to the attention of the Bridge Design Engineer and/or District Construction Engineer.

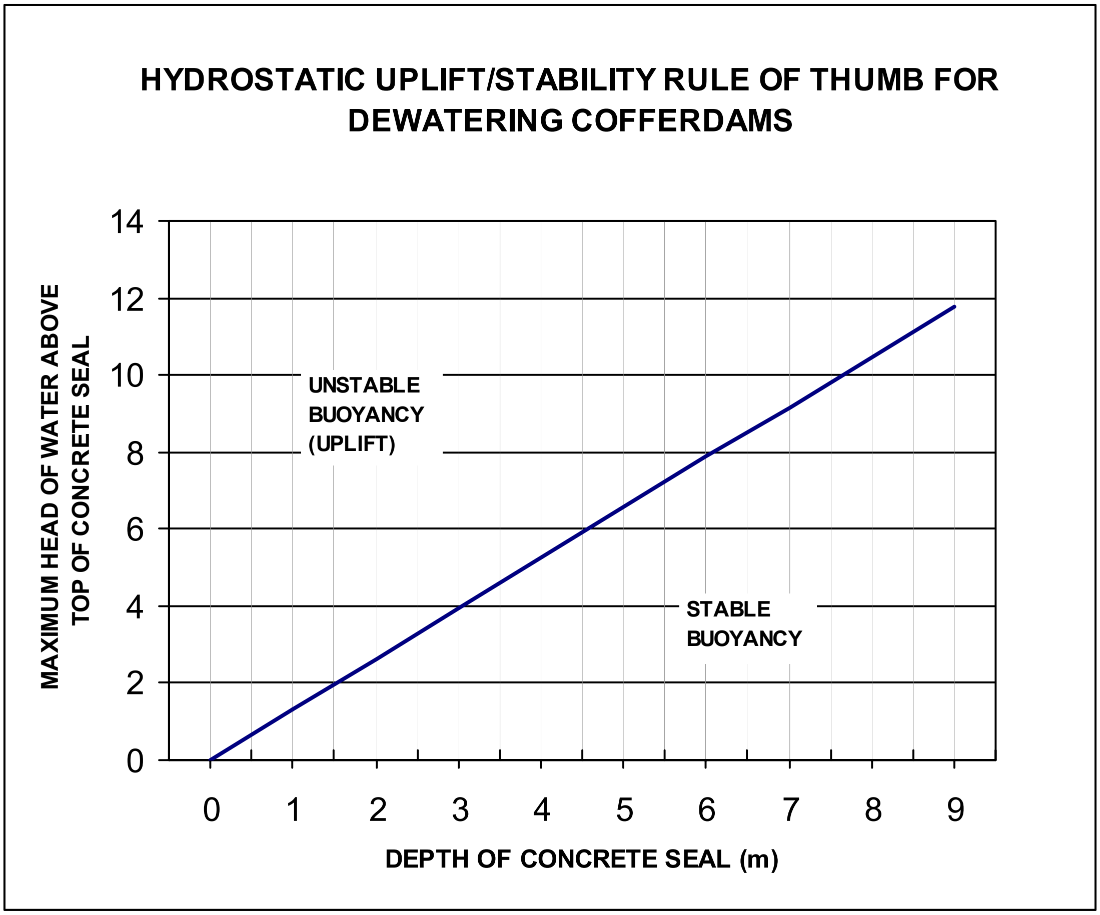


Figure 5-3 (M): Cofferdam Uplift Graph (Metric)

<u>Figure 5-3(M) Assumed Values:</u>

Unit Weight of Concrete	**2310 kg/cubic meter**
Unit Weight of Water	**1000 kg/cubic meter**

The weight of the sheeting and the framing and the friction of the seal against the sheeting are not considered

Figure 5-3(M) balances the hydrostatic uplift pressure of the water with the weight of the concrete and any case that is close to the unstable buoyancy side of the line should be brought to the attention of the Bridge Design Engineer and/or District Construction Engineer.

SECTION 504 - BRIDGE EXCAVATION

504.1 - GENERAL

This item involves the excavation of earth and rock material for the construction of bridge substructures, footings and seals. The contract administrator should contact the survey section, Bureau of Highway Design, for initial bridge control layout.

504.3 - CONSTRUCTION OPERATIONS

Common Bridge Excavation is a final pay (F) quantity item. This means that the quantity does not need to be measured, but shall be paid as the estimated bid quantity. However, (per the 9/2004 109.11 supplemental specifications) enough information should be gathered before any excavation begins so that a quantity could be calculated if it appears that the actual quantity is going to differ with the estimated bid quantity by more than 25%. Before any excavation operation is begun, the contract administrator should take sufficient original cross-sections to allow excavation quantities to be computed as simply and as accurately as possible. Cross sections for small structures often can be laid out by using centerline of bearing, channel, or roadway as a base line. Care should be taken to obtain cross sections at points of zero excavation and points of full width excavation as shown in Figure 5-4. Base lines, stations of cross-sections, and limits of excavation should be plotted on the record plans. In most cases this data will be for information only. However, if it becomes clear that there is a gross error (greater than 25%) between the actual and estimated bid quantities, then this information must be plotted and used to calculate the actual quantity of common bridge excavation.

When possible, the base line should be laid out along the principal axes of the structure in order to reduce the number of intermediate sections. More than one base line can be used when the additional base line would simplify the cross-sections. Figure 5-5 depicts an abutment with a skewed, elongated, stepped wing wall and a flying wing end that uses an additional base line and match line.

After cross-sections have been taken, batter boards should be set by the contractor to provide ready reference to lines and grades. The contract administrator should check the alignment and grade of batter boards after they have been constructed and braced properly. Unnecessary errors have often resulted from movement of poorly constructed batter boards.

The contract administrator should not permit large unsupported holes to be dug if nearby buildings, utilities, bridge structure units, or sloping ground surfaces may be affected. It may be necessary to modify adjacent slopes or support the sides of the excavation to protect adjacent structures and provide a safe working area.

During excavation operations, the contract administrator should inspect the material carefully at successive levels to ensure that it corresponds with the boring logs. Soil conditions may vary considerably in a given area. Should the material differ substantially, the contract administrator should determine whether the bearing capacity is adequate. If in doubt, the contract administrator should contact the District Construction Engineer and the Soils Engineer at the Bureau of Materials and Research. Refer to safety requirements in the contract for shoring and/or sheeting requirements.

In conjunction with Item 503, the underwater inspection of completed excavation to rock surface condition is conducted by independent divers. When this inspection is needed, the contract administrator should contact the District Construction Engineer.

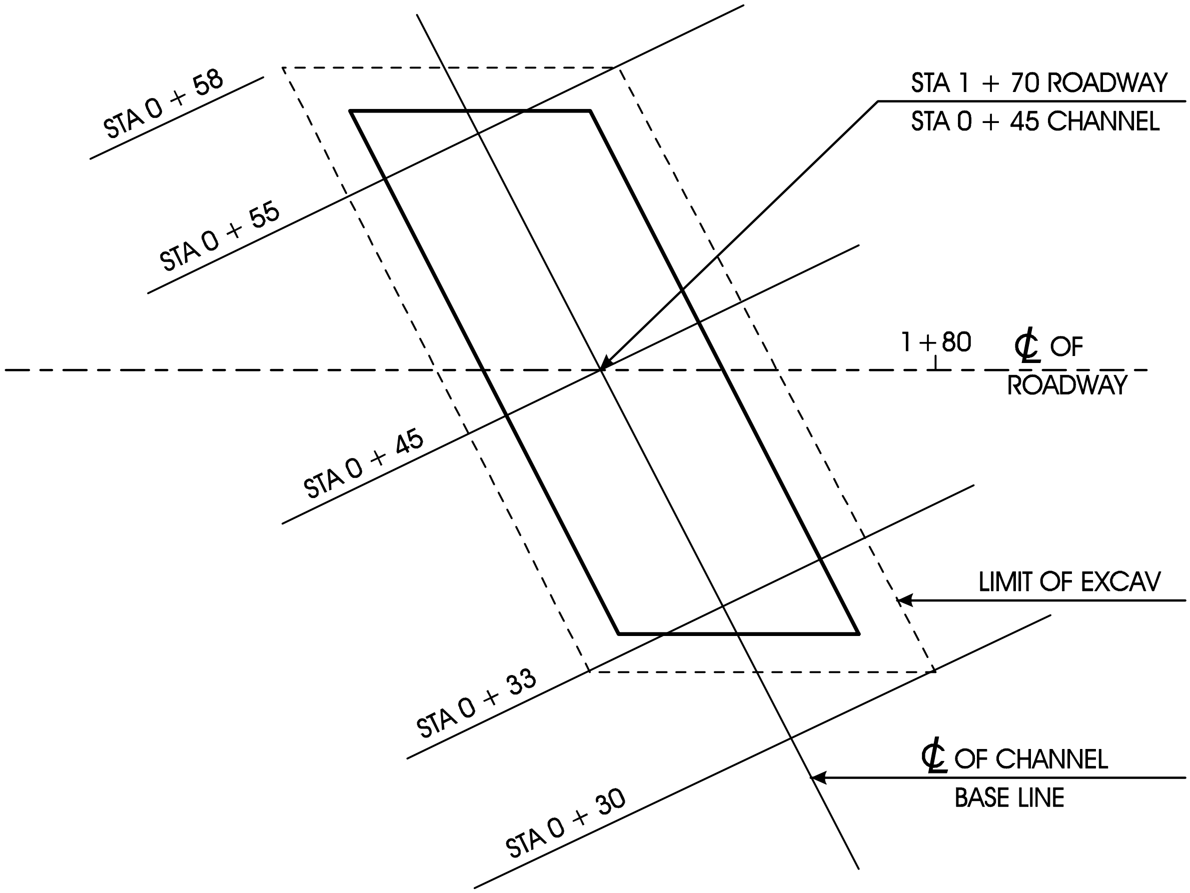


Figure 5-4: Bridge Excavation Survey

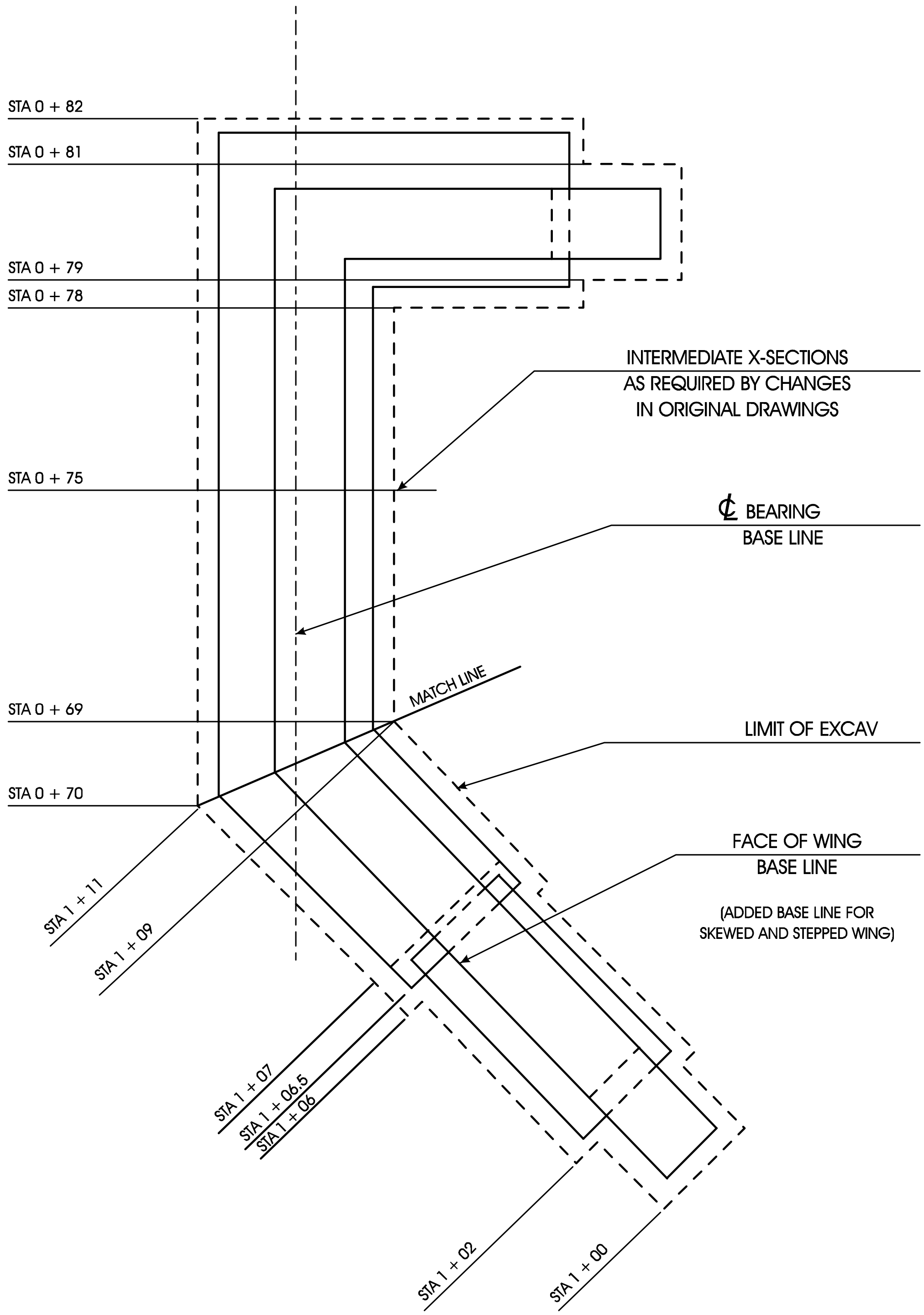


Figure 5-5: Bridge Excavation Survey Using an Additional Baseline

SECTION 506 - SHEET PILING

506.1 - GENERAL

Sheet piling is used to prevent scouring under structures; protect shore lines from moving water; or provide a bulkhead. Sheet piling ranges from simple wood planks and light gauge sheet metals to heavy sections made of structural steel members.

506.3 - CONSTRUCTION OPERATIONS

A. Layout:

The contract administrator should inspect the layout of the line that the piling will be driven on. Once the contract administrator is satisfied with the layout, an appropriate location for an offset line should be determined where it will not interfere with equipment performing the driving operation. Location of the control line should be discussed with the contractor's superintendent, and once it is set, it will be the contractor's responsibility to see that this agreed upon location is maintained.

B. Measuring:

In order to measure the cutoffs, paint marks should be placed approximately three feet (1 m) from the top of each pile before driving. Then the in-place length and cut-off lengths can be computed and recorded. Marking can usually be done well ahead of driving so that the inspector is free to keep track of the driving.

SECTION 508 - STRUCTURAL FILL

508.3 - CONSTRUCTION OPERATIONS

The plans should be closely followed relative to the minimum depth of excavation and the dimensions to which the structural fill is to be placed. Follow the notes on the plans or as stated in any addendums relative to placement of adjacent embankments in conjunction with the structural fill item.

Preparation of the area for structural fill is accomplished under bridge excavation. See Section 504 for more information. Generally, crushed gravel is the material used for Item 508 unless otherwise specified.

After the structural fill is in place, the contractor must keep the structural fill dewatered to maintain the maximum density until the footing concrete has been placed. To accomplish this, a simple ditch around the perimeter of the structural fill can be made to feed into a sump pump located at one corner of the excavation outside the limits of the footing and formwork.

During winter construction, the contractor must protect the structural fill from freezing until the concrete has been placed and properly backfilled.

SECTION 510 - BEARING PILES

510.1 - GENERAL

The pile, as a structural member, is used to transmit the load of a structure through a fluid or stratum of low bearing value to one of more adequate capacity; provide stable foundations in areas subject to scouring action; consolidate loose granular soils to some degree by driving high volume wedging action piles; anchor structures against uplift or overturning; and act as protective devices on piers in the form of fendering.

In the design stage, consideration is given to the use of batter piles when unstable soils are encountered. If vertical piles go down through an unstable soil and then come to refusal with only comparatively small penetration into the firm material, they have very little lateral stability. The weight of the approach embankment behind the abutment may be sufficient to cause the abutment either to tilt or to move bodily toward the stream. The unstable material flows under the pressure of the fill, moving the piles with it. The amount of movement will depend upon the soil condition and length of piles, but cannot be predetermined. When a critical soil condition is encountered as described above, and the contract administrator questions the plan pile configuration and design criteria, the observations that led to the questions should be discussed with the District Construction Engineer for evaluation.

A.Piles function in one of three ways:

1. The end-bearing pile acts as a supporting member to transmit a load through a semi-fluid or soft strata to hard material or rock.

2. A friction pile transmits the load to the soil throughout its entire length by the friction of the soil against the pile.

3. The compaction pile is intended to compact relatively loose soil by high volume wedging action.

510.2 - MATERIALS

A. Timber:

Timber piles, though rarely used, shall conform to ASTM D 25, Class B, clean-peeled and shall be preservative treated.

B. Precast Concrete:

Precast concrete piles are steel reinforced members that are cast and thoroughly cured before driving. Precast piles include conventionally reinforced piles and prestressed concrete piles. Both types are available in a number of different cross sections. Prestressed piles are more prevalent due to their greater resistance to damage during driving. In driving precast concrete piles, care must be taken to cushion the top of the pile from the direct impact of the hammer blow. Pile cushions for concrete piles should have the required thickness determined from a wave equation analysis but should not be less than 4” (100 mm). A new plywood, hardwood or composite wood pile cushion that is not water soaked should be used for every pile. The cushion material should be checked periodically for damage and replaced before excessive compression (more than half the original thickness), burning, or charring occurs. Wood cushions may take only 1000 to 2000 blows before they deteriorate. During hard driving, more than one cushion may be necessary for a single pile. The cushion must give enough protection to prevent local damage to the pile, without absorbing too much of the energy of the blow. Indifferent handling of piles may cause incipient cracks to form. These cracks may open up during driving or may even spall and "powder" to such an extent as to seriously lessen the strength and life of the pile. Precast piles should not be shipped from the plant or driven until they have obtained sufficient concrete strength to withstand handling and driving stresses. If the precast piles have been allowed to dry after curing, they should be wetted at least 6 hours before being driven and shall be kept moist until driven.

C. Cast-in-Place Concrete

Cast-in-place concrete piles are made by forming a hole in the soil and filling it with concrete. There are two general types of cast-in-place concrete piles:

1. <u>Shell:</u> Either a steel shell, which is heavy enough to be driven without a mandrel, is used or a light steel shell is driven with a mandrel, which is later removed. The shells for piles cast-in-place shall be carefully checked after driving for water tightness and deformation due to the driving of adjacent piles. A mirror for reflecting sunlight into the shell is the most common method for this check. On cloudy days, a flashlight may be lowered into the shell. Any water contained in pile shells should be siphoned out immediately prior to placing the reinforcing cage and concrete. If shells are not going to be filled immediately they should be covered and re-examined again prior to the placement of concrete.
2. <u>Shell-less:</u> The shell-less type of pile is made by driving a light steel shell with a mandrel or an earth auger.

D. Steel:

Steel H-piles are the most common piles in use in New Hampshire due to the general soil conditions. Of all the materials used for foundation piles, steel is the only material that has an ultimate strength in compression comparable to that of hard rock. Therefore, H-piles are used effectively in point bearing applications. In certain types of soil conditions steel piles may also rely on the "skin friction" between the pile and the soil to support a portion of the load. Steel pipe piles are used for applications where lateral strength is required in all directions. Closed-end piles are often driven to refusal and subsequently filled with concrete.

510.3 - CONSTRUCTION OPERATIONS

The contract administrator has many duties to perform during a pile driving operation, and consequently should become thoroughly familiar with the plans and specifications. The contractor shall submit a plan depicting the type of driving equipment, energy ratings of the hammer, and proposed lengths and sizes of the piles to the Bureau of Materials and Research Soils Engineer through the Bureau of Construction so that a minimum blow count can be calculated using the wave equation formula. This blow count will be given to the contract administrator as a requirement to be met to ensure the load capacity has been achieved without overdriving and possibly damaging the pile.

A dynamic pile test is often included in the contract as a special provision. The Bureau of Materials and Research will come out to the site and place instrumentation on a pile to evaluate driving stresses. This information will be used to adjust or verify the wave equation information.

A. Driving (General):

1. Penetration and Bearing Requirements:

   - Penetration: Plans usually show an estimated tip elevation or the notation that piles shall be driven to refusal on bedrock. Action of a given pile section being driven by a specific hammer into bedrock material of varying degrees of hardness or decomposition will have to be evaluated at the work site. The borings should be studied carefully to determine the nature and characteristics of the bedrock; the nature of the overburden above the bedrock; and the reason(s) for specifying practical refusal instead of a specific bearing value.

   - Bearing Requirements: For most projects, piles will be driven to develop a computed bearing of not less than the design load stated on the plans. Plans and specifications for large bridges often require load tests on various piles. The main purpose of a load test is to verify results indicated by the bearing formula. The first load test should be made early in the pile driving operation before any piles have been cut off.

2. <u>Location of Piles</u>: The contractor should lay out the individual pile locations and D.O.T. project personnel should check them for conformance with the plans. The layout is usually completed by installing roadway centerline and centerline of bearing on batter boards with a transit and then by use of string lines and tapes, measuring off each pile location. All piles, when driven, must be in line and in their true position. The contract administrator should strive for perfection in this respect (within reason), but in no case can deviations exceed those allowed in the specifications.

3. Preparation of Piles for Driving:

   When piles arrive on the project, they should be inspected by the contract administrator. In the case of epoxy coated piling, it may be necessary to touch-up scratched areas. Timber piles should be checked thoroughly for compliance with specifications. During inspection, piles should be marked every 1 ft (0.3 m) for blow count determination, thus preparing them to be placed in the leads for driving.

   - Timber piles are susceptible to damage during driving, particularly under hard driving conditions. To protect the pile against damage, the following procedures are suggested:
     - Cut the butt of the pile off square and chamfer it so that the hammer strikes evenly on the butt. The chamfered butt must fit the driving cap or hammer base, where used. If crushing, brooming, or splitting occurs, in addition to chamfering the butt, wrap the top end of the pile with 10 or 12 turns of heavy wire at a distance of about one diameter below the head of the pile. An alternative to wrapping is to clamp two half-rings of ½” (13 mm) steel around the butt. If a hole is bored in the butt of the pile, use double wrappings.
     - The pile is usually pointed by sharpening the tip to the shape of a truncated pyramid; the blunt end may be 4” to 6” (100 to 150 mm) square. The length of the point may be from one-and-a-half to two times the diameter of the tip. In extremely hard driving, steel points or shoes may be used to protect the tips of the piles. Shoes

of various sizes are available from manufacturers of pile driving equipment. Tips will be used only when specified on the plans.

- Steel H or pipe piles are driven with caps specially designed to fit over the tops of the piles. The points of steel H-piles may be reinforced by adding welded or riveted plates. Thickness of the web and flange may be built up to 2 ½ to 3 times the original for a height of 2 ½ or 3 times the width. At the mill, or in the field, the lower ends of steel pipe piles may be fitted with flat plate ends, pressed steel points, or special cast or fabricated steel shoes for open-end driving.
- Pile Design lengths should be checked against borings prior to the contractor ordering them to avoid unnecessary splices or large quantities of cut offs. Wherever possible, steel piles should be ordered from the mill cut to the required length, in order to avoid field splicing or cutting. Note, however, that any holes burned in the piles for lifting purposes must be plug welded if they should fall below the bottom elevation of the concrete foundation. As an alternative, holes may be burned in the neutral axis of the pile and in this case do not require plugging.

4. <u>Driving Procedure</u>: The procedure for driving piles consists of four basic steps.

    - The pile driver is brought into position with the hammer and cap at the top of the leads.
    - The pile line is lashed to the top of the pile and then the pile is raised in the leads. Next, the tip of the pile is placed in the proper position.
    - The pile is centered under the pile cap and the pile cap and hammer are lowered to the top of the pile. If a drop hammer is being used, the cap is unhooked from the hammer.
    - In the case of a drop hammer, the hammer is then raised and dropped to drive the pile. The driving is started slowly with a drop hammer, raising the hammer only 2”-4” (50 to 100 mm) for each blow until the pile is firmly set. The height of the fall may then be increased to a maximum of 10 ft (3 m) or so. Blows should be applied as rapidly as possible, in order to keep the pile moving.

    With steam or pneumatic hammers, operating pressures should be restricted until the pile is firmly set. The pressure may then be increased to the recommended value. For diesel pile hammers, the recommendations of the manufacturer should be closely followed.

5. <u>Positioning the Guiding Piles</u>: When piles are driven on land, and where a reasonably liberal tolerance is permitted as to plumbness and position of the top of the pile, the position of each pile may be marked with a stake. In driving steel H-piles by this method with suitable leads used, the final position of the top of the pile should be within about 2” (50 mm) of its theoretical position, and it should be plumb within about 1” (25 mm) per 5’ (1.5 m) of length. It is generally desirable to systematically offset positioning stakes from the final desired pile position, so that the stake will remain in place as the tip of the pile is maneuvered into position.

A single guide frame may be constructed to hold the piles in position during driving where closer tolerances are required. Occasionally, it is required that piles be driven with very close tolerances. In such cases, it is necessary to erect an accurate guide frame at or close to the ground (or water) level with a second frame 20 to 30 feet (6 to 9 m) or more above it, depending upon the length of the piles and the distance they extend above ground or water level. In this manner the piles are held plumb and exactly in position. Driving accuracy within ¼” (6 mm) of exact position is possible by this method.

Whenever possible it is desirable to drive the vertical piles first, often located in the back row of the pile location plan of the footing. After these are driven, the guide frame, or template as it is more commonly referred to, can be placed. Usually a pile is laid on the ground along the line of battered piles. After it is properly located it is then braced back to the previously driven vertical piles. This template facilitates the placement and driving of the battered piles.

When piles are to be driven in water, one of several methods may be used to mark the desired pile positions. When a number of bents are to be constructed, a stake is placed at each abutment approximately 6” (150 mm) from the pile centerline. A wire rope is stretched between the two stakes and a piece of tape or cable clip is fastened to the rope at each pile position.

When a floating pile driver is used, a frame for positioning piles may be fastened to the hull.

6. <u>Precautions During Driving</u>: Very careful watch must be kept during the driving of a pile in order to avoid damage to the pile or pile hammer.

The pile driver must be securely braked or fastened down to prevent movement during driving.

The hammer hoist line must be kept slack at all times while the pile is being driven, so that the full weight of the hammer rests squarely on the pile. It is essential that the fall of the hammer be in line with the pile axis; otherwise, the head of the pile may be damaged severely, the hammer damaged, or much of the energy of the hammer blow lost.

Bouncing may be due to the use of a hammer that is too light. Usually however, it occurs when the pile has met an obstruction or the pile has penetrated to a solid layer. When a double-acting hammer is being used, bouncing may be due to the use of too much steam or air pressure. When the last few blows of the hammer will not drive the pile more than the computed penetration per blow for the required bearing value of the pile, penetration has stopped because of an obstruction or refusal. <u>Further driving will inevitably damage or the “cripple” the pile</u>. It must be remembered that this penetration applies only when the driving is being done with the right size hammer operating at rated energy.

If the lack of penetration seems to be due to an obstruction, it may be small enough that 10 or 15 blows of less than maximum impact will drive through the obstacle and start the pile moving. If the pile has encountered a firm stratum, this fact may be detected by driving a few other piles nearby.

7. Driving Bearing Piles in Groups: When piles must be driven in closely spaced groups, keep the following facts in mind.

   In a sand or gravel deposit, the soil must be compacted or displaced an amount equal to the volume of the pile. If the deposit is quite loose, the vibration of pile driving frequently results in considerable compaction of the soil. The surface of the ground between piles then may subside or "shrink". Careful watch must be kept, lest this subsidence cause damage to the foundations of nearby structures.

   If piles are driven into dense sand and gravel deposits, some heave of the ground may take place. Clay soils are relatively incompressible under the action of pile driving. Hence, a volume of soil equal to that of the pile usually will be displaced. This results in heave of the ground between and around the piles.

   The driving of a pile along side those previously driven, frequently will cause those already in place to heave upward. Such cases may be detected by taking level readings on the tops of piles previously placed. Piles that heave greater than 1/2" (13 mm) must be redriven to firm bearing.

   Displacement of soil by the pile may create sufficient lateral force to move previously driven piles out of line. Serious damage may result to shells driven in the construction of cast-in-place concrete piles or to "green" cast-in-place piles of the shell-less type.

   The sequence of driving piles in groups should be as follows:

   - Driving should progress from an area of high resistance to low resistance toward a stream, or down slope. This minimizes shoving previously driven piles out of place when succeeding piles are driven.
   - Outer rows in the group should be driven first if the piles derive their principal support from friction. Inner rows are driven first if the piles derive their support from point bearing.

8. Battered Piles: The presence of battered piles dictates the sequence of driving all piles. The contractor must decide the driving order so that the battered piles can be driven at the correct angle and location without creating obstacles for the remainder of the operation. The driving area, particularly if it is structural fill, must accommodate the driving equipment and crane. The intended batter angle must be rigidly held.

9. Pile Driving Inspection: The inspector should continuously observe the driving operations, and at the same time, relate the rate of movement of the pile under the driving force to the exploratory data provided on the plans. The inspector should not make the error of allowing driving to be terminated at an elevation above the "minimum penetration" elevation when the tip of the pile has hit a thin, hard layer that may overlay a softer material. When driving piling of any type, the inspector shall keep the contract administrator advised of any unusual changes in driving performance.

Driving should be continuous whenever possible. Interruptions of driving will falsely show on the driving record as an indication that the pile is nearing refusal. Upon the continuation of driving, the pile may not move even under maximum stroke due to friction forces that have "grabbed a hold" of the pile while it's been sitting (particularly if you get back on a pile the following day). The effort must be continued, however, because the pile will break loose and continue to penetrate. Extra care must be taken to make sure that the hammer starts off with a smaller stroke on such a pile. The friction forces that have built up will resist the initial stroke of the hammer causing a higher stroke rebound than desired. This increased load due to the higher hammer stroke could very easily cripple the pile. Whenever pile driving is interrupted, the driving record for that pile should so indicate, so no misinterpretation of the data will occur to indicate a hard driving spot for this pile.

10. Driving Through Obstructions: Frequently, obstructions are encountered below the surface of the ground during pile driving operations, particularly when piles are driven in the industrial and commercial areas of older cities. They are a matter of considerable concern since they may prevent a pile from penetrating far enough to provide adequate load-carrying capacity.

    When an obstruction like a rotten log or timber is encountered, 10 or 15 blows of the hammer may cause the pile to break through it. If the obstruction cannot be breached by this method, the pile may be withdrawn, a small explosive charge lowered to the bottom of the hole and the obstruction blasted out of the way.

    Many times, an obstruction such as a log can be driven through by pulling the pile and cutting the leading edges to a chisel shape.

11. Underwater Driving: It is sometimes desirable to drive piles underwater, rather than by the use of a pile follower. Both double-acting and single-acting air or steam hammers are suitable for driving underwater when properly handled and rigged. The recommendations of the hammer manufacturer in preparing and rigging the hammer should be followed. Special attention must be given to proper lubrication of the hammer.

12. Effects of Driving on Adjacent Structures: When piles are to be driven for a new foundation along side an existing structure, care must be taken to see that the existing structure is not damaged. Shrinkage or heave of the ground around the new piles may cause serious damage.

    In such cases, careful records on the levels of ground in the area and the behavior of adjacent foundations must be kept. If piles are to be driven behind a retaining wall, the pressure on the wall may be greatly increased. The increase in pressure may be due to consolidation of a granular soil by vibration while a plastic soil may actually be forced against the wall. Also, if battered piles are to be driven within a cofferdam, the cofferdam should be installed at a sufficient width to avoid conflicts with the piles as they are driven outward underground.

    In critical locations, special methods of placing piles - such as jetting or jacking - may have to be used.

13. Straightening: Piles should be carefully examined during driving for proper orientation and alignment. This is particularly important for the initial 10-20 feet (3-6 m) of penetration. Pile driving should be immediately terminated if any significant

twisting or bending of the pile is observed. In such a case, the cause of the problem should be evaluated. Removal and relocation of the pile or pre-boring may be necessary. In general, procedures to twist or force a pile into proper alignment should be avoided since this can cause overstress in the pile. Piles should be straightened as soon as any misalignment is noticed during driving to meet job specifications. The greater the penetration along the wrong line, the more difficult it is to get the pile back into plumb. The following are commonly used methods of realigning a pile:

- The use of a block and tackle, with the impact of the hammer jarring the pile back into line. See Figure 5-6 for an illustration of this.
- A jet may be used in combination with the method described above.
- It is sometimes possible to force timber piles, which have been driven along the wrong line, back into alignment by the use of an alignment frame constructed of heavy timber and bolts.
- A vibratory pile driver can be used to partially extract and realign the pile.

Realignment of steel H-piles frequently must include twisting of the individual piles to bring the webs of the piles into the proper position.

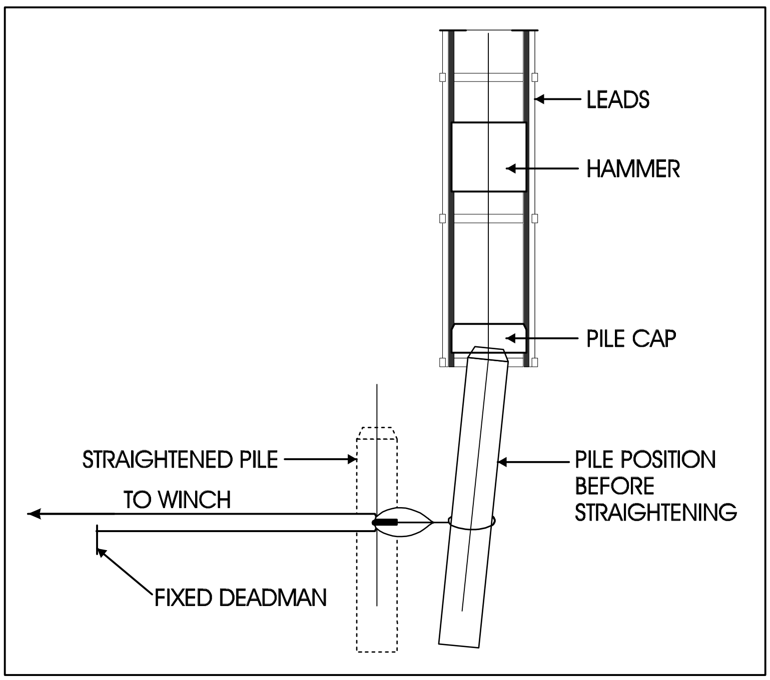


Figure 5-6: Pile Straightening

14. <u>Length Adjustment:</u> For timber piles, length adjustment is a relatively simple matter. Normally, after driving, the butts of piles are 2 or 3 feet (600 or 900 mm) above the desired finished elevation. The piles are then sawed off at the desired level. For timber piles in a bent, a good method is to nail sawing guides across all piles in the bent. Timber piles that are too short are easily spliced. The splice must be strong and stiff in order to develop the necessary resistance to bending. Sleeve pipe joints may be used for splicing and usually are made of 8” to 10” (200 or 250 mm) steel pipe cut in 3’ (1 m) lengths. Contact ends must be carefully cut to give full contact, and pile ends are trimmed to fit snugly in the pipe. A flat transverse bar through the sleeve between

the abutting pile ends will keep the sleeve in place during driving. Bolting timber or steel splice plates may also be used.

Steel piles are adjustable in length by cutting or splicing. Splices can be made by bolting, riveting, or welding. When welding two sections together they must be butt-welded. Figure 5-7 below indicates a butt-welded 100 percent splice. It is customary to order piles that are to be spliced cut to mill tolerances. Milled ends are unnecessary since scarfing of the piles in preparation of butt-welding is done in the field by torch-cutting.

Shells for cast-in-place concrete piles are easily adjusted in length by cutting or welding. Adjustment in length of precast concrete piles is somewhat more difficult. The Special Provisions or the Bureau of Bridge Design should be consulted for details of the procedure.

15. Welder Qualification: Welders shall meet the qualifications listed under standard specification 550.3.6.2. Welders responsible for welding pile splices shall be groove or "G' qualified. Fillet "F" welds are not sufficient when welding pile splices.

For additional welding information see section 550.3.F of this manual.

16. Pile Driving in Cold Weather: It is possible to conduct pile driving operations in severe cold, even though the ground is frozen. Frost up to 2' (600 mm) in thickness can be broken through successfully by driving a heavy pilot pile or a heavy casing.

Another way to "break" frost is to thaw the ground by spreading unslaked lime over the area to a depth of 3" to 4" (75-100 mm); covering the lime with snow; then covering the snow with tarpaulin; then covering the tarpaulin with snow again. This method has been found to melt through a layer of frost 3 ft (1 m) thick in 12 hours or so.

Earth augers are capable of drilling holes in frozen ground and may be used to aid pile driving. The driving of piles for bridge foundations may be somewhat easier during cold weather, in that it may be possible to work through sound river ice. Holes cut through the ice act as guides for the piles.

Special instructions of the manufacturer concerning operation of steam, air, or diesel pile hammers during cold weather should be followed. Contractors will often spray ether into the ports of a cold diesel hammer to help get it started.

17. Concrete Damage Due to Pile Driving: The contractor should coordinate its pile driving and concrete placement operations in such a manner that no damage or displacement will occur to concrete masonry in any existing substructure unit as a result of pile driving operations in any other unit.

To the extent practicable, all pile driving within a substructure unit should be completed before any concrete is placed in that unit. Should it become necessary to drive piling or conduct other construction operations that might adversely affect freshly placed concrete, including blasting and demolition of existing structures, check with the District Construction Engineer. A period of 72 hours is often required between placing concrete and starting or resuming nearby pile driving operations.

18. Pulling Piles: It is frequently necessary to pull piles from the ground in order to reuse them. This is particularly true of steel sheet piles used in cofferdams, temporary retaining walls, and so on.

The most effective way of pulling a pile is by using a vibratory pile extractor while exerting a strong upward pull. Commonly, specially designed double-acting extractors are used, but it is possible to rig a pile hammer in an inverted position to pull piles.

B. Equipment:

1. Pile Hammers: Six types of pile hammers are now recognized: drop, single-acting, double-acting, differential-acting, diesel, and vibratory.

   Single-acting, double-acting, and differential-acting pile hammers depend upon steam or compressed air for motive power. A general term for these is steam or pneumatic hammers. There are two types of diesel hammers: open-end and closed-end. One other type of hammer, which is in limited use, is the sonic hammer.

   The contract administrator, knowing the name and model of the pile hammer that the Contractor expects to use, can find the data for it in the tables under the PDI-Products GRLWEAP Hammer File Listing found online at website. http://web.pile.com/pdi/products/grlweap/hammer.asp?company.

   Manufacturers of air, steam, or diesel type driving hammers designate each size of each type of their hammers by a number and the rated energy output of the hammer. This designated energy rating is usually the maximum energy that the hammer is capable of producing as determined by the manufacturer by use of a formula or actual energy measurement. The operating speed of mechanical hammers is extremely important. The speed must be maintained to produce the energy intended. The Materials & Research Soils Engineer or the Bridge Design Engineer must approve the type and size of the hammer prior to commencing the operation.

   - Drop Hammers: A drop hammer basically is a block of metal that falls on the end of the pile. Standard drop hammers are made in various sizes and weights. For example, Vulcan Iron Works of Chattanooga, TN makes several models that range in weight from 500 to 3000 lbs. (230 to 1360 kg). Hammers are supplied with a round steel pin for use in hoisting. A triangular die, grippers, and trip devices can be supplied when desired.

     Similarly, the Eagle Iron Works, Des Moines, Iowa, manufactures several standard drop hammers which range in weight from 1500 to 7000 lbs. (680 to 3175 kg). Hammers and accompanying follow blocks are made of tough semi-steel. Hammers are supplied with leaded-in steel pegs for fastening the follow block sling.

     The distance through which the hammer falls can be varied within the limits of equipment available. The drop hammer is the simplest type of pile hammer. However, it is comparatively slow in comparison with other types.

     When manufacturer's literature is unavailable, drop hammers must be weighed before any piles are driven. The drop hammer stroke should be carefully measured. This can be done by taping a piece of rope or rag around the hammer line at the height above the hammer for the drop desired. The operator can then gauge the drop with reasonable accuracy. Short fast strokes of the hammer are more efficient than long slow strokes.

- <u>Single-Acting Hammers:</u> In general terms, a steam or air hammer consists of a stationary cylinder and a moving part (ram), which includes the piston and striking head. A single-acting hammer is designed so that the ram is raised by steam or air pressure and drops by gravity.

  This type of hammer is marked by a heavy ram, low impact velocity, and a comparatively small number of blows per unit of time. The most powerful hammers made are of this type. A variety of single-acting hammers is available. For example, Vulcan supplies five standard models of this type. Typical of Vulcan hammers is model No. 2, which has a 3000 lb (1360 kg) ram, a rated striking energy of 7225 ft-lbs (9.8 kJ) per blow, at a rate of 70 blows per minute. The McKiernan-Terry Corporation of Dover, New Jersey, also manufactures several hammers of this type, including the S-20, which has a 20,000 lb (9,070 kg) ram, a total striking energy of 60,000 ft- lbs (81.3 kJ) per blow, and a rated speed of 60 blows per minute.

  Because of its operating characteristics, this type of hammer is generally used where heavy piles; piles of low compressive strength (such as precast concrete); or piles in stiff clay, compact gravel, or other dense soils are to be driven.

- <u>Double-Acting Hammers:</u> The piston of a double-acting pneumatic or steam hammer is raised by the motive fluid and, in addition to the gravity fall, the piston is forced down by pressure. Double-acting hammers are characterized by high frequency of blows - from 90 to 100 blows per minute. They are extensively used for driving sheet piles and bearing piles through common soils. The high frequency of blows results in faster driving by keeping the pile moving and preventing the buildup of soil friction.

  A disadvantage of this type of hammer is the relatively high impact velocity, which may result in excessive deformation of the head of any low compressive strength pile.

  A variety of sizes of double-acting hammers are available from manufacturers. Models range from extremely light hammers with rams that weigh less than 100 lbs (45 kg) up to heavy units. One large hammer of this type – the McKiernan-Terry model 11B3 – has a ram that weighs 5000 lbs (2268 kg), an 18” (483 mm) stroke, and a frequency of 95 blows per minute.

- <u>Differential-Acting Hammers:</u> A differential-acting pile hammer is a variation of the double-acting pile hammer. The piston and cylinder are designed to reduce the size of prime mover required. Ram weights are greater than for comparable double-acting hammers in order to obtain lower impact velocity. Frequency of blows is greater than for comparable single-acting hammers.

  Vulcan Iron Works manufactures seven different models of open-type, differential-acting pile hammers of different sizes and weights. Typical of these is hammer 80C. Striking parts for this hammer weigh 8000 lbs (3630 kg), the rated striking energy per blow is (33.1 kJ) per blow, and the frequency of blows is 111 per minute. Vulcan's model DGH Portable Pile Hammer is a small differential-acting hammer that has a gross weight of 850 lbs (386 kg).

- Diesel Hammers: Diesel hammers are of two types.
  - One type has an open upper end, whereby the ram is unrestricted in its rebound and is visible above the body of the hammer. The height of the rebound is taken as the length of the stroke for the following blow. Under normal driving conditions, the height of rebound will increase as the resistance of the pile to driving increases. There is a force exerted on the pile by the explosion of the charge of fuel, and likewise there is a loss in the kinetic energy developed during the fall of the ram due to the cushioning effect the explosion of the fuel has on the impact of the ram. It is assumed that the energy gain of one is about equal to the energy loss of the other, and therefore the energy output for this type of hammer in foot-pounds (Joules) is the product of the weight of the ram and the length of stroke.

    Measurement of the length of stroke is made by observation, reading on a graduated rod attached to and extending above the hammer body or shell, the height to which the top of the ram reaches in its rebound.

    It is important to note that blow counts determined by the wave equation analysis apply only when the hammer is operating at the proper (often maximum) stroke. The inspector will develop a sense of full desired stroke based on visual obstruction of the cylinder, the tone of the ring and the time between blows.

  - For the other type of diesel hammer, the ram operates in a cylinder that is closed at the top and the upstroke of the ram traps and compresses air in the bounce chamber, which is the space in the cylinder above the top of the ram. The energy stored in the compressed air is imparted to the ram on the downward stroke. The output energy of the hammer is designated the equivalent energy measured in foot-pounds (Joules). This equivalent energy consists of the product of the weight of the ram and the length of its stroke; and the force due to the compressed air in the bounce chamber, which in turn is equivalent to an additional height in the fall of the hammer.

    As the resistance of the pile increases, the force of explosion of the diesel fuel, acting on the ram in its upstroke, increases, and the increased energy of the ram increases the energy stored in the compressed air chamber, which in turn provides an increase in the force imparted to the ram at the start of its downstroke. Thus, it is evident that the energy output of this hammer, within the limits of its rated energy output, will increase as the resistance of the pile being driven increases.

When determining the bearing capacity of a driven pile, it will be necessary, at the time of the count of the blows of the hammer and measurement of set of pile, to determine the equivalent energy of the hammer. From a gauge attached to an air hose that is connected to the bounce chamber, readings of the air pressure are made and these pressure values are converted to equivalent energy in joules by means of charts prepared for the hammer and furnished with the gauge.

In the event the resistance of the pile during the driving is sufficient to produce an air pressure in the bounce chamber which, as read by the gauge and converted by chart into equivalent energy, is of a value greater than the manufacturer's rated energy for the hammer, the manufacturer's rated energy for the hammer should be used in the bearing formula.

- Sonic Hammers: Sonic and vibrating hammers have effected a major advance in pile driving.

  The sonic hammer has the advantage of operating with almost total lack of noise and vibration, which is most desirable in urban areas.

  However, the rate of penetration varies little with changes in soil conditions and the speed of penetration makes it difficult to determine the bearing capacity of friction piles. Therefore, these hammers lend themselves best to sheet-pile and end-bearing pile applications until some means of determining the accurate driving energy is developed. These types of hammers should not be used for the driving items without prior approval of the contract administrator and the Bureau of Materials and Research. Load-bearing piles driven with these hammers must be "tested" through the use of another type of hammer for which blow count energy can be determined.

2. Power for Hammer: Power for operation of a steam or pneumatic pile hammer is supplied by either a steam boiler or an air compressor.

   Steam boilers in general use are oil-fired. These boilers have generally replaced coal-fired boilers because of simpler fuel stocking and easier operation. Fast-steaming, lighter-weight generators - such as the McKiernan-Clayton Steam Generator - designed specifically for use in pile driving operations are available.

   Portable gasoline or diesel engine-powered air compressors are commonly used for pile driving. More than one compressor may be necessary to meet the requirements of the largest hammers.

   Diesel pile hammers are self-contained, supplying their own motive power.

Selection of proper equipment for hammer operation is an important factor in controlling pile driving costs. Factors of importance include adequate capacity of boiler or compressor, adequate working pressure at the hammer, losses in connecting air and steam lines, and so on. The contractor may consult the manufacturer of the hammer for specific recommendations as to power requirements of the hammer in question.

3. Selection of Hammer: Many factors enter into the selection of the proper type and size of hammer to be used under a specific set of conditions (this is true in the selection of all pile driving equipment). Proper equipment selection will cut the contractor's job costs and increase job profit.

General factors of importance in hammer (and other equipment) selection include the type, length, weight and shape of pile; soil conditions at the job site; and engineering specifications pertaining to the job.

Specific selection should be based upon manufacturers' recommendations and previous on-the-job experience. The following general principles are of value.

- If a drop hammer is being used, experience dictates the use of a relatively heavy hammer and low fall. Such a combination will normally produce a greater penetration of the pile per blow with less damage to the pile head or cushion. The hammer should weigh at least as much as the pile it is to drive. Effective results are frequently obtained with a hammer weighing approximately twice as much as the pile. In the case of concrete piles, properly cushioned, the provision of about 25 lbs. (17,600 kg) of hammer weight for every square inch (square meter) of hammer weight per square foot (meter) of pile section has been found satisfactory. In order to quickly accelerate the hammer during the hoisting period, an engine capable of exerting a line pull in pounds (newtons) of approximately twice (19.6 times) the weight in pounds (kilograms) of the hammer is necessary.
- A single-acting pile hammer may be selected for driving conditions previously stated. As with a drop hammer, best results are usually attained if the ram of the hammer is at least as heavy as the pile being driven.
- Double-acting steam or pneumatic hammers may be used for driving any type of pile. As previously stated, their principal use is in driving sheet piles and bearing piles in common soil. Because of their high impact velocity, the pile head must be adequately protected during driving.
- Recommendations for differential-acting hammers are essentially the same as for single-acting hammers.
- Diesel hammers (most commonly used at present) offer obvious advantages of mobility and usefulness in remote or inaccessible areas, since they are not dependent on either steam generators or air compressors. McKiernan-Terry states that their model DE-30 offers most economical driving with 1 to 3 ton (900 to 2700 kg) piles at bearings from 40 to 90 tons (356 to 800 kN); model DE-20, with ½ to 2 ton (450 to 1800) kg piles at bearings from 25-60 Tons (220 to 535 kN).

- Some manufacturers may, for a particular hammer, list both a maximum rated energy and an average working energy. For uniformity in procedure in determining that a hammer satisfies the specified requirements for size, the manufacturer's rated energy of the hammer will be used and if more than one energy rating is listed for a particular hammer, it will be the maximum energy rating.
- The energy output of a hammer may not, during the driving of a pile, be the same as the rated energy. For a double or differential acting air or steam hammer there should be sufficient air or steam pressure at the hammer to operate the hammer at the number of blows per minute required for a given energy rating. The energy output will vary if the number of blows per minute deviates from the designated number.

Although the contractor selects pile driving equipment that is deemed adequate to drive piles to the necessary depth and bearing without materially damaging the piles, it is advisable for the contract administrator to be familiar with the power plant, hammer, caps, leads and other elements used in driving.

The Specifications may contain the minimum energy rating of an air, steam, or diesel hammer required to drive a pile to a designated bearing. One purpose of this is to provide sufficient energy for driving the pile to the required bearing without incurring a set or penetration per blow that is so small that when it is used in the formula for determining the bearing power of the pile it may give unreliable results.

4. <u>Attachments for Pile Hammers:</u> A variety of attachments are available for use with pile hammers in order to drive different types, sizes, and shapes of piles. These include those known as base attachments, driving heads, pile caps, helmets, anvils, followers, pants, and others. Terminology is not standard throughout the industry. Use of available attachments to fit a certain hammer permits more efficient and economical driving. In some cases, they are a necessity if the work is to be accomplished properly.

   The contractor may consult the manufacturer of the pile hammer for recommendations pertaining to attachments for use in a given set of conditions. Figure 5-7 below illustrates selected pile driver attachments.

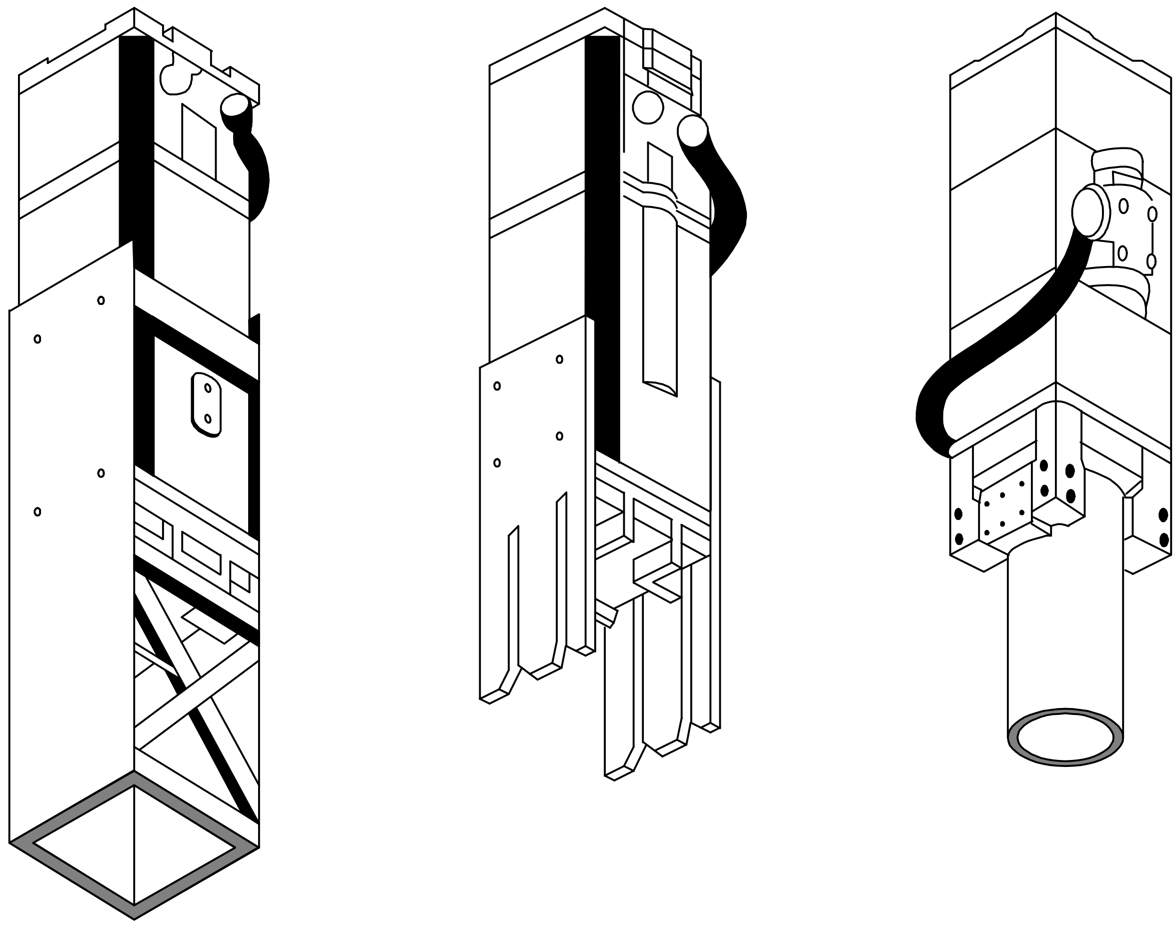


Figure 5-7: Special Pile Driver Attachments

5. Pile Leads: Pile driving leads serve as tracks along which the hammer runs, and as guides for positioning and steadying the pile during the first part of driving. Leads manufactured from steel are available in a number of combinations, usually in sections 5' (1.5 m) to 20' (6 m) long. Long leads may be fabricated to special order, or job built by the contractor.

   Pre-fabricated leads include the following types:

   - Fixed leads are used to drive piles vertically or at a fixed batter. The leads are rigidly attached to the pile driver or crane. See Figure 5-9 (F) for an illustration of fixed leads.
   - Swinging (hanging) leads are suspended from the boom point of the driver, either by a line and bail attached to the top of the leads or by boom-point adapters that are bolted to the boom-point and to the leads. Typically, the bottom of the leads is set (toed) at ground level to hold the pile's position and the leads are then plumbed or battered as required by the crane boom. See Figure 5-9 (S) for an illustration of swinging leads. Figure 5-8 illustrates the assembly of the driving cap and the drop hammer in the bottom of the leads.

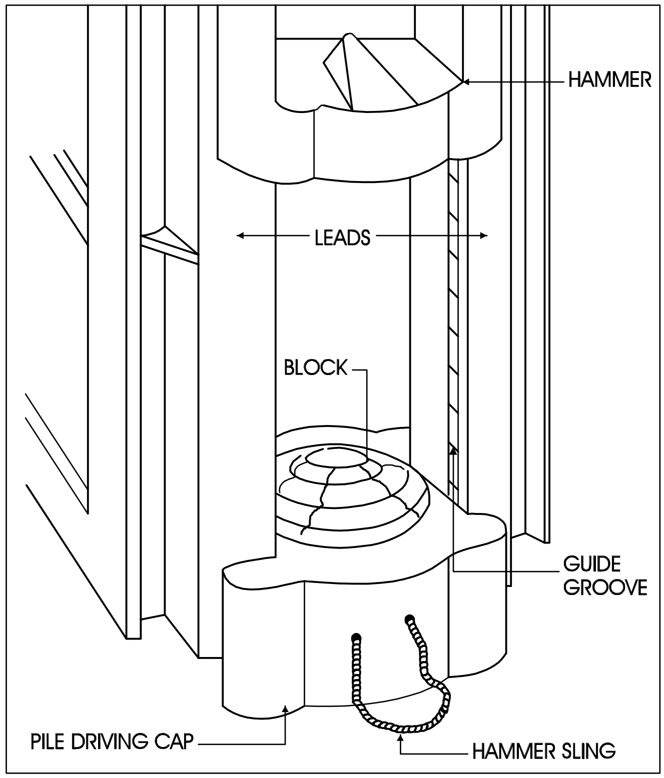


Figure 5-8: Assembly of Drop Hammer and Driving Cap in Bottom of Leads

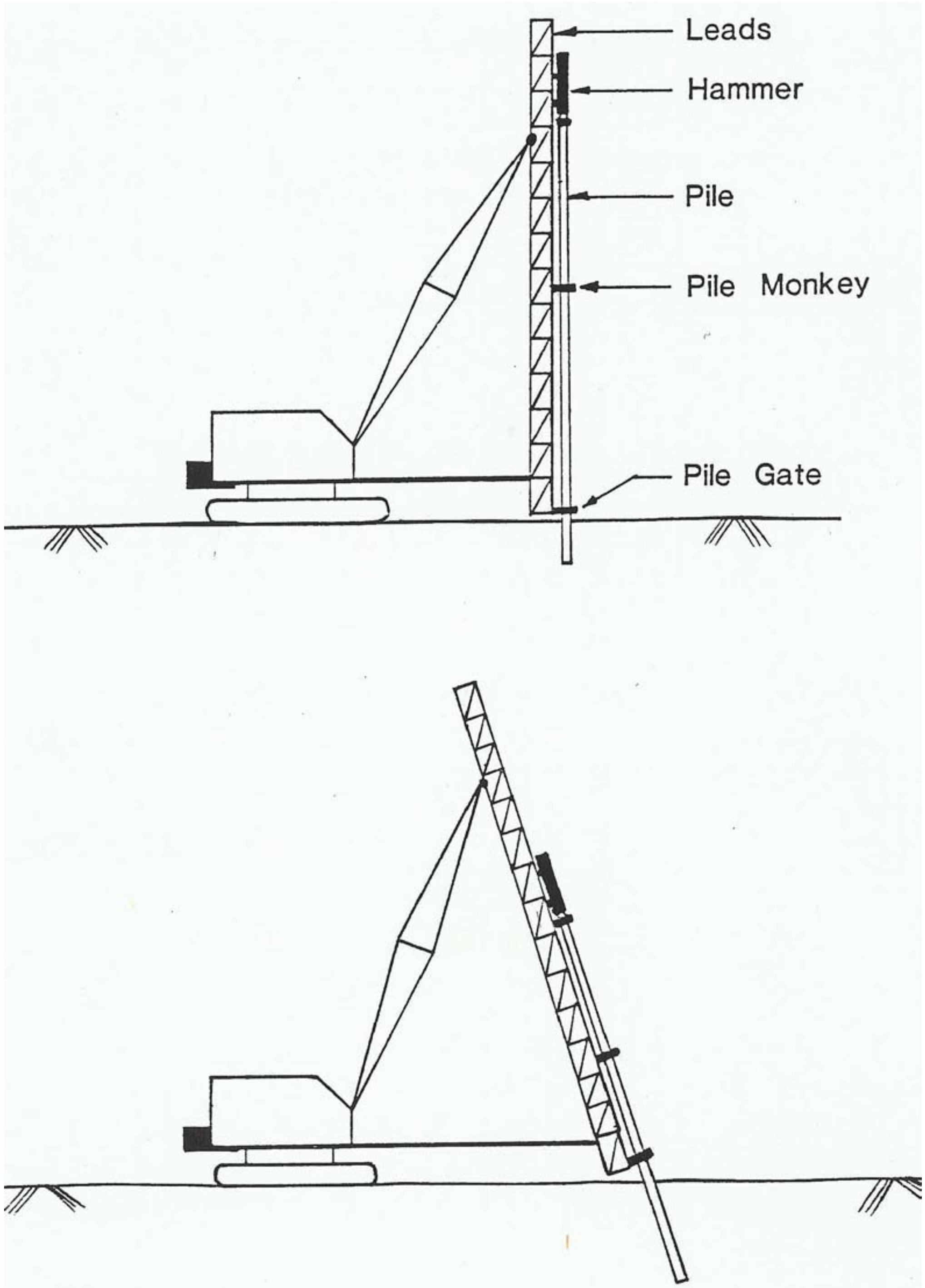


Figure 5-9 (F): Fixed Leads

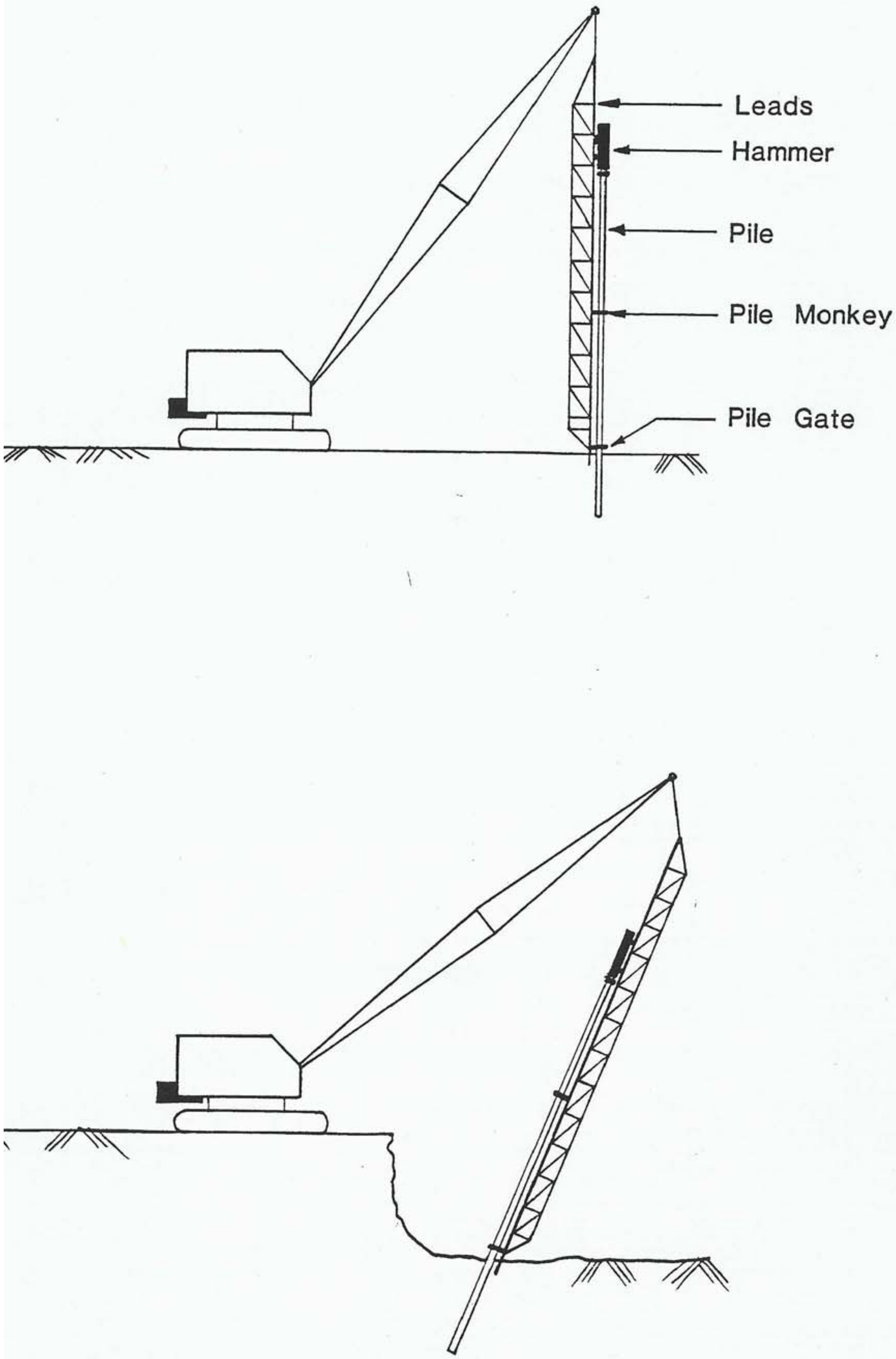


Figure 5-9 (S): Swinging (Hanging) Leads

- Underhung vertical or batter leads hang beneath the boom-point and are held in position by lead braces that extend from the bottom of the leads to the base of the pile driver or crane. Lead braces are adjustable to attain, within limits, any desired batter.

- Extended leads extend above the boom point. Leads of this type may be arranged to drive vertical or batter piles. One type, the McKiernan-Terry extended 4-way batter leads, holds the hammer aligned with the pile at vertical and one fore, aft, right and left batter by attaching the lead to the bottom point and, through a bottom brace and moonbeam, to the cab of the crane. Leads of this type carry their own set of lead sheaves. Figure 5-10 illustrates the McKiernan-Terry extended 4-way batter leads.

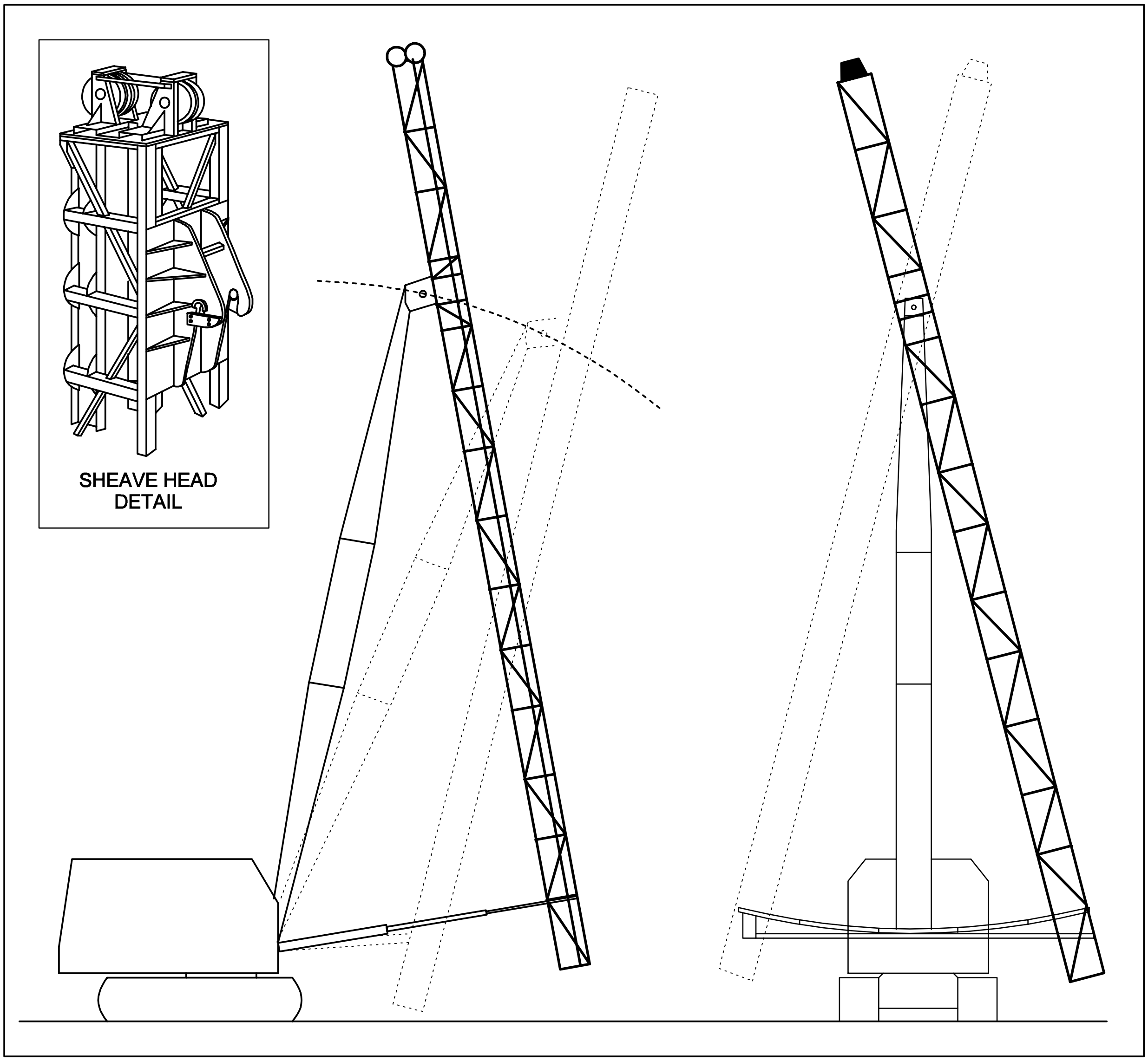


Figure 5-10: McKiernan-Terry 4-Way Batter Leads

6. Attachments for Leads:

- Extension Above Boom Point: An extension above boom point is a design variation that can be applied to 6:1 and 3:1 fore and aft batter leads. It permits the use of a shorter crane boom with a correspondingly greater crane capacity than is obtainable with a full length boom for the specified lead length. It consists of a special boom hook section and necessary extension sections. A sheave head is built into the uppermost section. See 5-11 for a diagram of a lead attachment extension above boom point.

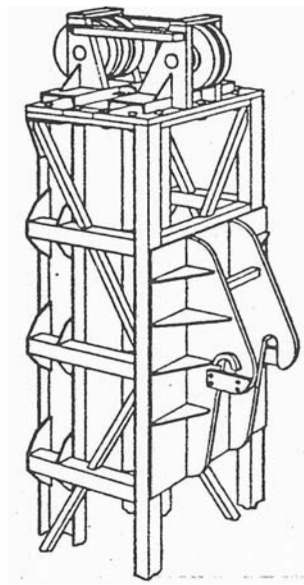


Figure 5-11: Extension Above Boom Point

- Hairpin: A hairpin provides an easy means of entering and removing a hammer from the leads. It can be blocked at any point in the leads permitting the hammer to slip below it and out of the leads without removing the angle-iron guides or dropping the hammer in a hole. A hairpin also provides a means of using a narrow hammer in leads that are also used for a wide hammer. See Figure 5-12 for a diagram of a hairpin lead attachment.

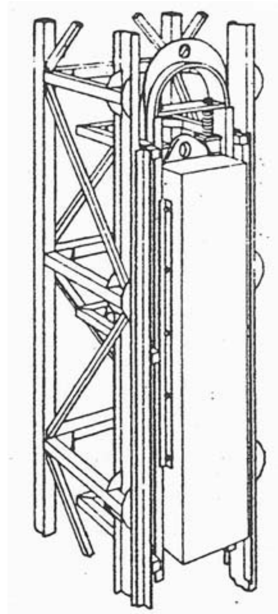


Figure 5-12: Hairpin Lead Attachment

Telescope: A telescope is used to provide an extension below the lead to permit supported operation of the hammer below grade. See Figure 5-13 for an example of a telescope lead attachment.

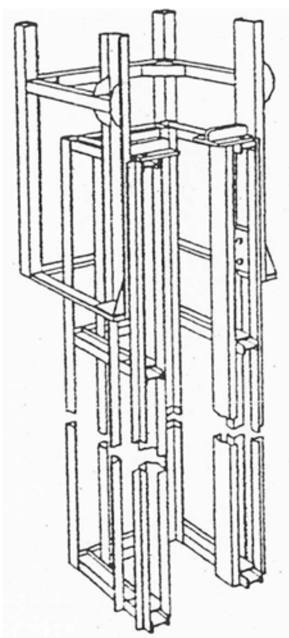


Figure 5-13: Telescope Lead Attachment

6. Jetting equipment: In a typical case, jetting equipment consists of standard steel pipe and pipe fittings that are made into a jetting assembly. Flexible water hose and couplings connect the jetting assembly to a pump. The jetting assembly may be handled by the same rig used to handle the piles.

   Jetting pipes are usually from 2 ½" to 3 ½" (65 to 90 mm) in diameter. For use in gravelly soils, water pressure should range from 100 to 150 psi (690 to 1030 kPa). For sand, water pressure from 40 to 60 psi (275 to 415 kPa) is generally adequate. Piles may be jetted by attaching pipes to the sides of the pile as shown in Figure 5-14.

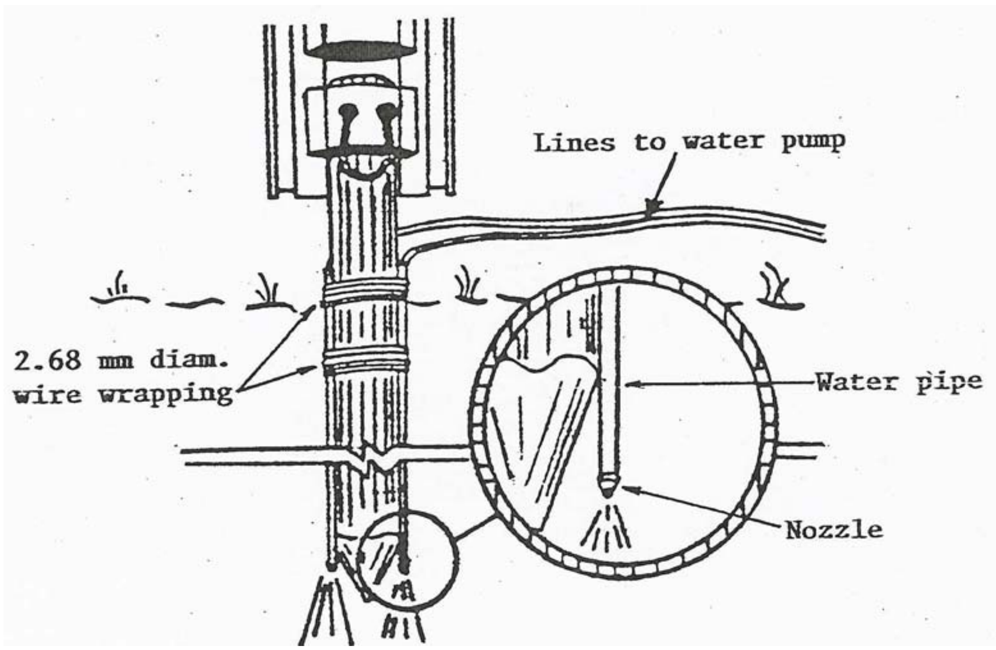


Figure 5-14: Suggested Method for Jetting Piles

7. Pile Extractors: Pile extractors are available from manufacturers of pile hammers. They are essentially specially designed double-acting hammers operated by steam or air. Extractors are furnished in various sizes suited to the type of pile to be pulled and the lifting capacity of the crane. It is always desirable to exert as much upward pull on the pile as possible. Therefore, an extractor should be chosen that is capable of withstanding the maximum pull that can be exerted by the crane. Various attachments are available for use with the extractor in pulling different types of piles.

   If desired, certain double-acting hammers can be rigged in an inverted position with a wire rope sling and used as pile extractors. Generally speaking, the use of this method is not as efficient as is the use of an extractor. The following table lists the specifications of two types of McKiernan-Terry extractors. Figure 5-15 illustrates an extractor.

Table 5-1: Specifications of McKiernan-Terry Extractors

Size Number	**E2**	**E4**
Net Weight of extractor and attachment – lbs (kg)	2600 (1 179)	4400 (1 996)
Weight of ram – lbs (kg)	200 (91)	400 (181)
Bore - in. (mm)	7 (178)	9 (229)
Stroke - in. (mm)	3 (76)	3 (76)
Energy of blow – ft-lbs (J)	700 (949)	1000 (1 356)
Strokes per minute	450	400
Width, overall - in. (mm)	25 (635)	26 (660)
Depth, overall - in. (mm)	19 (483)	22 (559)
Length, overall – in. (mm)	100 (2 540)	125 (3 175)
Diameter of pile clamp bolt - in. (mm)	2 3/8 (60)	2 3/8 (60)
Width of standard pile clamp - in. (mm)	6 (152)	6 (152)
Air consumption, actual - CFS (cubic meters per second)	400 (0.19)	550 (0.26)
Boiler power - H.P. (W)	30 (294 285)	35 (343 333)
Hose connection - in.(mm)	1 ½ (38)	1 ½ (38)
Maximum crane pull - Tons (metric tons)	50 (45)	100 (91)

Steam or air pressure should not exceed 125 psi (862 kPa) gauge pressure.

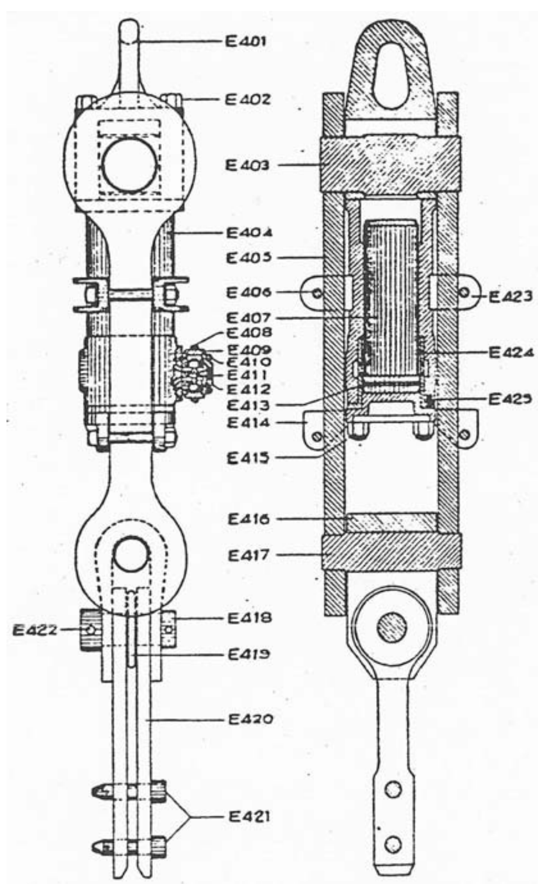
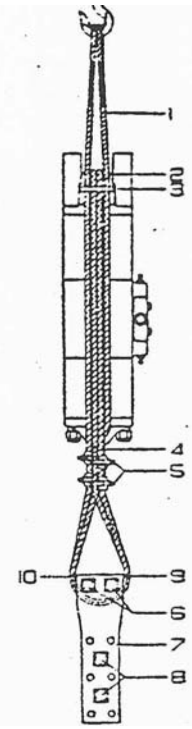


Figure 5-15: McKiernan-Terry Extractor

The following diagram illustrates a cable pulling rig for using a pile hammer to extract piling. The rig shown was devised for use with McKiernan-Terry Nos. 5, 6, and 7 hammers.



CABLE-PULLING RIG PARTS

Part No.	Name	No. Req.
1	Crane Sling	1
2	Grooved Saddle Block	1
3	Retaining Ring	1
4	Pile Sling	1
5	Clips	8
6	Pile Sling Bolt & Nut	2
7	Pile Clamp	2
8	Pile Bolt & Nut	2
9	Half Sheave (single)	2
10	Half Sheave (double)	1

Figure 5-16: Cable Pulling Rig for Pile Extraction

C. Wave Equation Analysis:

Traditional dynamic formulas, such as the Engineering News Record Formula, do not provide accurate predications of actual pile capacities and they provide no information on stresses in the piles. The one-dimensional wave equation analysis has eliminated many shortcomings associated with the pile driving process. The term wave equation is a name applied to several computer programs that use a finite element to model dynamic pile behavior.

The wave equation analysis uses wave propagation theory to monitor the longitudinal wave transmitted along the pile axis when it is struck by a hammer's impact. As the ram impact occurs, a force pulse is developed in the pile that travels downward toward the pile tip at a constant velocity that depends on the pile's material properties. When the force pulse reaches the portion of the pile that is embedded, it is attenuated by soil frictional resistance along the pile. If the attenuation is incomplete, the force pulse will reach the pile tip and a reflected force pulse, which is governed by the soil tip resistance, is generated. The pile will penetrate into the soil when the peak force generated by the ram impact exceeds the ultimate soil resistance at the pile tip.

The wave equation analysis provides two types of information:

1. It provides a relationship between pile capacity and driving resistance. The user inputs data on soil side shear and end bearing and the anaylsis provides an estimate of the set [in/blow (mm/blow] under one blow of the hammer. By specifying a range of ultimate pile capacities, the user obtains a relationship between ultimate pile capacity and penetration resistance [blows per 1" (25 mm) or blows per 10" (0.25 m).

2. The analysis also provides relationships between driving stresses in the pile and penetration resistance.

   The wave equation analysis enables the user to develop curves of capacity versus blow count for different pile lengths and to use these in the field to determine when the pile has been driven sufficiently to develop the required capacity. The wave equation also provides results for blow count versus stroke for a particular pile capacity. The analysis is used to select the right combination of driving equipment to:
   - Ensure that the piles can be driven to the required depth and capacity.
   - Design the minimum required pile section for driving.
   - Minimize the chances of over stressing the pile.
   - Minimize driving costs.

   A static analysis is performed in order to provide input data for the wave equation analysis. The input data generated by the static analysis includes pile length, load transfer distribution, and the ultimate load capacity.

   The contractor's pile driving equipment summary should be provided to the Bureau of Materials and Research Soils Engineer at least 30 days prior to pile driving so that a wave equation analysis can be conducted. The Soils Engineer should also visit the site during the initial stages of pile driving to verify the applicability of the wave equation analysis driving criteria.

D. Pile Dynamic Load Testing:

Technical services and equipment for dynamically testing piles are available through the Geotechnical Section of the Materials and Research Bureau. This work should be scheduled and coordinated through the Soils Engineer.

The testing equipment for dynamic testing includes these components:

- A pair of strain transducers mounted near the top of the pile
- A pair of accelerometers mounted near the top of the pile.
- A pile driving analyzer (PDA).

The pile driving analyzer monitors the output from the strain transducers and accelerometers as the pile is being driven and evaluates the data as follows:

- The strain data, combined with the modulus of elasticity and cross-sectional area of the pile, gives the axial force in the pile.
- The acceleration data, integrated with time, produces the particle velocity of the waves traveling through the pile.
- The acceleration data, double integrated with time, produces the pile set per blow.

These dynamic measurements are used to evaluate the performance of the pile hammer; calculate pile installation stresses; determine pile integrity; and estimate static pile capacity. Dynamic test results can be further evaluated using signal matching techniques to determine the relative soil resistance distribution on the pile as well as representative dynamic soil properties for use in wave equation analysis.

In general, dynamic load testing should be conducted on the first piles driven for a substructure to evaluate the driving system efficiency, to evaluate the dynamic pile capacity and to check pile stresses. The dynamic load test should also be conducted on selected piles as necessary, particularly if there is a question of pile damage.

E. Pile Static Load Testing:

Static load tests may be required on certain projects and are usually specified in the contract. Static load tests should be conducted early in the construction phase and if possible before pile lengths are ordered. A static load test may also be required if the driving conditions encountered during construction are not consistent with the anticipated subsurface conditions and if dynamic measurements cannot give a reasonable interpretation of the pile capacity.

The main purpose of the load test is to verify that the actual pile response to load corresponds to the response assumed by the designer and that the actual ultimate pile load is not less than the computed ultimate load used for design. The static load test can result in foundation cost savings, particularly on a friction pile project.

ASTM D-1143 should be followed for compression load testing of piles. This procedure involves applying a compression load to the pile and measuring movements. Most often, compressive loads are applied by hydraulica1ly jacking against a beam that is anchored by piles or ground anchors, or by jacking against a weighted platform. The load should be measured using a calibrated pressure gage and calibrated load cell. Axial pile head movements are usually measured by dial gages that measure the pile head movement relative to an independently supported reference beam.

The setup, monitoring, and interpretation of the load test should be coordinated through the Soils Engineer. Several methods used for pile load tests are shown in Figure 5-20 found at the end of this section.

F. Characteristics of H-Piles in Various Soils and when Driven to Rock:

The subject of soil mechanics and the relationship of this subject to foundation piles have been thoroughly discussed in a number of books and articles. We shall endeavor only to tie in a few of the broader principles to the specific subject of H-piles.

Soils are classified according to size by the U.S. Department of Agriculture, as follows:

Gravel.................................. larger than 0.08 in (2 mm) diameter

Sand...................................... 0 .002 to 0.08 in (0.05 to 2.00 mm)

Silt.. 0.001 to 0.002 in (0.002 to 0.05 mm)

Clay...................................... smaller than 0.0001 in (0.002 mm)

In natural beds, soil occurs, as often as not, as a mixture of two or more of the above classifications. Thus, we commonly see soil described as sandy clay, silty sand, clayey sand, etc. The problem is further complicated for the engineer designing a pile foundation by the fact that soils usually occur in layers of differing classifications, frequently with many layers at the site of a particular project.

A foundation pile driven entirely into soil derives its load-carrying capacity from two sources, namely, point resistance and skin friction.

1. Point resistance may be defined as the resistance of the soil to being pushed aside to make way for the pile. This action occurs at and around the end of the pile in the case of a pile with parallel sides. A similar action occurs elsewhere along the pile if the pile is tapered or it's an H-pile with special attachments.

2. Skin friction, as its name implies, is the resistance of the soil to sliding along the surface of the pile.

For the purpose of this section, soils and subsoil conditions shall be divided into four groups, and each discussed in relation to H-piles. These classifications are hard rock; shale, hardpan, marl, and soft rock; sand, gravel, sand and gravel; and clay.

1. H-Piles Driven to Hard Rock:

Steel is the only material used for foundation piles that has an ultimate strength in compression comparable to that of hard rock. This fact results in an entire field of foundation structures for which H-piles are predominantly suitable. For example, in the case of bridge piers founded on rock lying at great depth below water level, H-piles have been used in lengths close to 200 ft (60 m), carrying extremely high loads per pile. These piles, driven on close centers, are capable of taking advantage of the high bearing capacity of the rock in a manner not possible with other types of piles.

In a majority of cases, the surface of hard rock is covered by a thin layer, 4 inches (100 mm) or more in thickness, of soft disintegrated rock. The H-piles penetrate the soft layer, and become firmly seated on the hard rock. If the rock is overlain by hardpan, sand and gravel, stiff clay, or other hard material, the condition is very satisfactory for fixing the point of the pile for bearing on the rock. In all such cases, it is not necessary to incur the expense of reinforcing the point of the pile. In the occasional case of extremely hard ledge rock immediately overlain by very soft material such as mud, consideration may be given to reinforcing the point of the pile. A detail of suggested reinforcement for this purpose is shown in Figure 5-18 found at the end of this section.

2. H-Piles in Shale, Hardpan, Marl, and Various Soft Rocks:

No discussion of the mechanics of this class of material is necessary, since it is readily admissible that point resistance, or in some cases, point resistance plus skin friction over a short length of the pile will develop full bearing capacity. H-piles are particularly suitable in this situation because high load concentrations are possible. The smaller number of H-piles, each carrying a greater load than is feasible with most other types of piles, will produce economies not only in the piles themselves but in other directions.

The fact that H-piles can withstand sustained hard driving makes it possible to obtain deeper penetration in this class of material than is possible with piles of bulky cross-section. This is important, since it is possible to firmly fix and seat the piles.

3. H-Piles in Sand, Gravel, or Sand and Gravel:

The chief properties of sand, gravel, and sand and gravel, aside from the size of individual particles, are permeability, incompressibility (unless in a loose, uncompacted state), high coefficient of friction, and low cohesive strength. These properties are directly opposite to those of clay, which are dealt with in a subsequent section.

Since soils comprised of compact sand, gravel, or sand and gravel are incompressible to a high degree, the principal action at the point of the pile is to push aside the soil to make room for the pile. There is little compaction of the soil, and water is not squeezed out of the voids as in the case of clay. When driving ceases, the soil is already in a state of equilibrium, and the point resistance of the pile remains constant.

In compact sand, skin friction is not decreased by water lubrication during driving, as in the case of clay, since there is no tendency for the water to escape. When driving is discontinued, the pressure of the soil against the pile is about the same as during driving, and the skin friction resulting there from becomes an important source of load-carrying capacity.

H-piles are especially suitable for use in compact sand, gravel, or sand and gravel. Because these soils are highly incompressible, the displacement resulting from driving an H-pile, though such displacement is comparatively small, is ample to develop a high intensity of compressive stress both at the end point and along the sides of the pile, with correspondingly high values for point resistance and skin friction.

The resistance of sand and of gravel to being pushed aside by a pile is so great that it usually is impossible to obtain more than a small amount of penetration with the type of pile that has a solid cross-section and large bulk. In many instances, a certain minimum amount of penetration is just as important as load-carrying capacity. This is often so in the case of bridges where the river bottom is subject to deep scour, and it is imperative to drive the piles to sufficient depth to protect against such action. H-piles are in a class by themselves in such cases. They can be driven further than other types of piles because of their small displacement. Their ability to take extreme punishment allows them to stand being driven long after they have reached a depth where ample bearing capacity has been developed.

Adequate bearing capacity normally is reached in this class of soils with a moderate amount of penetration by the use of plain H-piles, and it is, therefore, unnecessary to modify the piles with any attachment. In occasional cases of very fine saturated sand, or where the sand is mixed with a minor percentage of clay, or of loose sand that is subject to compaction due to rearrangement of the particles under vibrating loads, the penetration of plain H-piles may be deemed excessive. It may become economically desirable at this point to increase the bulk of the piles with suitable attachments, commonly called lagging. This should be necessary for only a comparatively short distance 8’ to 12’ (2.4 to 3.6 m) and the lagging should be located near the top of the load-bearing stratum. If located near the bottom, it will have the effect of destroying the skin friction of the piles above the lagging. This is due to the transition from the bulky cross-section to the plain H-pile relieving the soil of the compressive stresses that produce skin friction.

Details of various kinds of lagging are shown in Figure 5-17.

4. H-Piles in Clay:

The chief characteristics distinguishing clay from other soils are the smallness of the particles; a high degree of impermeability and compressibility; a low coefficient of friction; and cohesive strength that may vary from 87 psi (4.8 kPa) or less for liquid clay, to 14 psi (96 kPa) for very stiff clay. These characteristics vary, generally speaking, with the moisture content. The engineering properties deteriorate as the percentage of moisture increases. This does not mean that the moisture content is a definite index to the properties of all clays, as two different clays with the same moisture content may differ decidedly as to properties.

It has been noted that clay is compressible. This is true to a far greater degree than in the case of sand or gravel. When pressure is applied, water is squeezed out of the voids, and the solid particles are pressed into closer contact. The water naturally escapes into the avenue of least resistance. Skin friction during driving of an H-pile in clay may be small. The soil, which has been forced aside, exerts pressure against the sides of the pile, but since the pile is in motion, the small coefficient of friction of the clay results in small frictional resistance. The coefficient of friction will even be reduced below its normal value by lubrication caused by water squeezed out of the clay by compression at the pile point and lateral compression due to vibration of the pile.

After driving is completed, however, the lateral pressure, which has been set up during driving, forces the soil particles into close contact with the comparatively rough surface of the pile, resulting in a strong bond. It is this bond that furnishes the chief means of transmitting a load from the pile to the soil. In many cases, the bond is stronger than the shearing resistance of the soil. This is demonstrated when an H-pile in clay is pulled. It is not unusual for the pile to come up with the spaces between flanges and web filled with cores of soil.

The great difference that may exist in clay between driving resistance and static resistance under load is well known. Everyone acquainted with pile driving is familiar with the phenomenon of a pile that drives easily in clay but after a period of rest sets up solidly. Upon redriving such a pile, the initial penetration per blow is considerably less than the final penetration for the original driving. In the case of plain H-piles, the static resistance almost always will be greater than what the driving resistance indicates because the skin friction increases from what may be a negligible value during driving to a substantial value after rest.

In hard, stiff clay containing only a small percentage of moisture, the compressibility will be small, and therefore the amount of displacement and compression required to develop its full capacity will be correspondingly small. When an H-pile is driven into such clay, it frequently happens that the soil trapped between the flanges and web becomes so hard due to compression that it grips the pile and is carried down with it. The pile thereupon becomes in effect a displacement pile, and the core of soil trapped on each side of the web performs the same function that lagging serves in softer soil. Under such conditions, plain H-piles will develop very satisfactory load-carrying values.

In the more plastic compressible clays, the bearing value of a plain H-pile may be small, and consideration must be given to the lateral compressibility of the soil in relation to the cross section and bulk of the pile. In such soils the proper use of H-piles may necessitate the attachment of suitable lagging to the piles to increase their displacement and thereby develop more fully the bearing capacity of the soil. This type of pile has an advantage over a conventional displacement pile where extremely long lengths are required, where there is lateral impact (as in water-front structures), where there is a long unsupported length, etc. The lagging should be attached throughout the depth of the load-bearing stratum, instead of for a yard (meter) or so as in the case of loose saturated sand. If the clay is overlain by soft soil of no bearing value, nothing is gained by lengthening the lagging to include such soft material. The lagging provides the large section area at the point of the pile which is necessary to obtain the degree of displacement and compression to develop the full resistance of the soil, and it also maintains the pressure along the sides of the pile necessary for skin friction.

G. Lagging:

Lagging is attached to an H-pile to increase its cross-section and thereby its displacement of the soil. The purpose of this is to increase the bearing value of the pile. Its use is considered only where the bearing value must be developed in plastic, cohesive soil, and occasionally in loose, uncompacted, fine sand. It is of no value where the pile is driven into compact sand, sand and gravel, hardpan, or soft rock,

If possible, comparative load tests of lagged and unlagged piles should be made to determine definitely whether the lagging is economical. The fact that an unlagged H-pile drives easily through plastic cohesive soil does not necessarily mean that it has no bearing value under such conditions. After driving ceases, the pile may set up a bond with the soil that will develop a satisfactory load-carrying capacity. For additional discussion of lagging, see the section covering H-piles in clay. One type of lagging is shown in Figure 5-17 below..

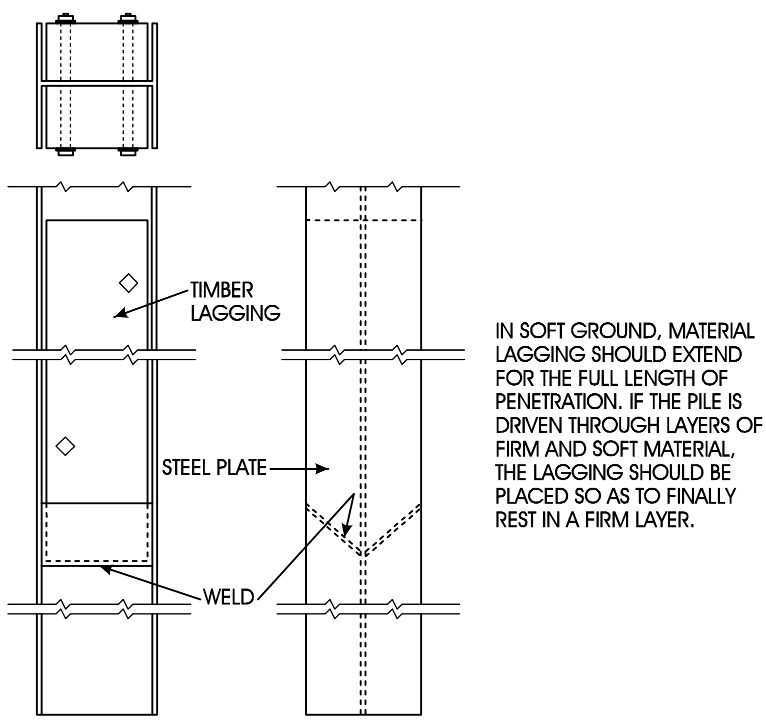


Figure 5-17: Lagging

H. Pile Point Reinforcement:

Pile point reinforcement never should be used where the piles must develop skin friction, since the enlarged point will greatly reduce the skin friction on the surface of the pile above the point. This limits the use of pile point reinforcement to cases where the piles are driven to hard rock. It is specified by some engineers to distribute the load over the rock in order to reduce the unit stress, although there is some question regarding the necessity for its use for this purpose, as discussed several pages before this under the section covering H-piles driven to rock. Another reason for its use is to protect the point of the pile from buckling when driven to hard rock. Here again, care against overdriving the piles usually will prevent serious buckling. Figure 5-18 below illustrates some types of pile point reinforcement. Although commercially available, pile points are routinely ordered and welded onto the leading ends of the piles. NOTE: The use of pile tips is decided in the design stage and is not generally a decision made in the field.

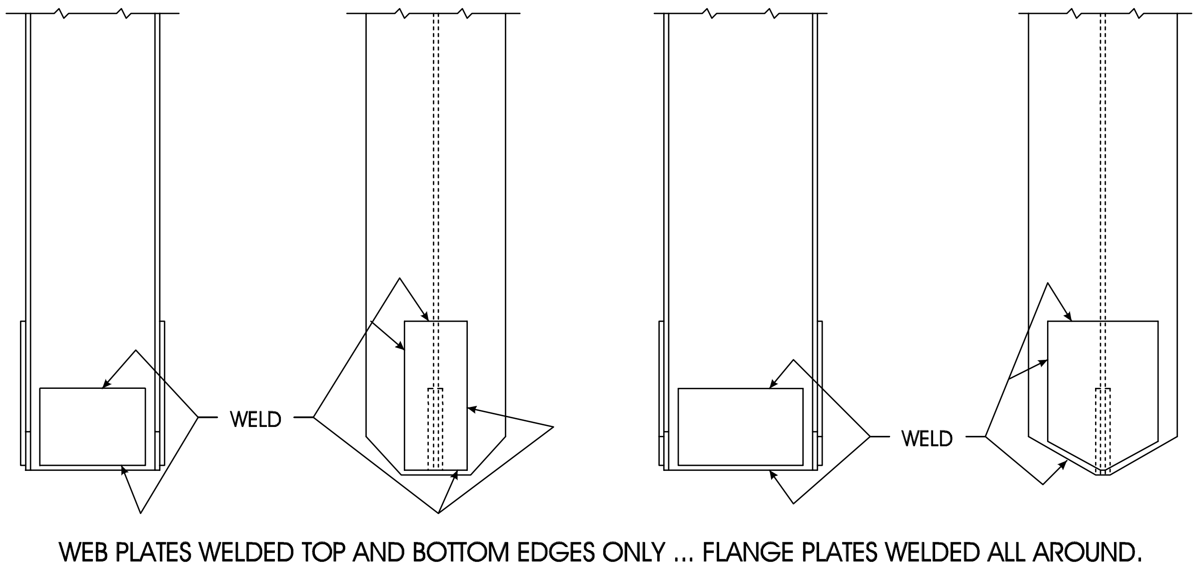


Figure 5-18: Types of Pile Point Reinforcement for H-Piles

I. Pile Driving Formula:

Generally speaking, where the size and nature of the project will permit, loaded test piles should be used instead of formulas for determining the safe load per pile. Engineers tend to keep the load per pile low where a formula must be depended upon due to the uncertainties inherent in the use of a formula. Substantial economies can often be affected by increasing the load per pile based upon the reliable information obtainable from properly conducted pile load tests.

Pile-driving formulas are particularly useful for small projects where the cost of load tests would be an important percentage of the total cost of the piles required for the project.

FORMULA FOR THEORETICAL ELASTIC SHORTENING OF PILE

$$\text{Shortening (e)} = \frac{\text{Load} \times \text{Length}}{\text{Area} \times \text{Modulus of Elasticity}} = \frac{\text{PL}}{\text{AE}}$$

$$\text{English: } e = \frac{\text{Load (pounds)} \times \text{Length (inches)}}{\text{Area (in}^2) \times (2.9 \times 10^7)}$$

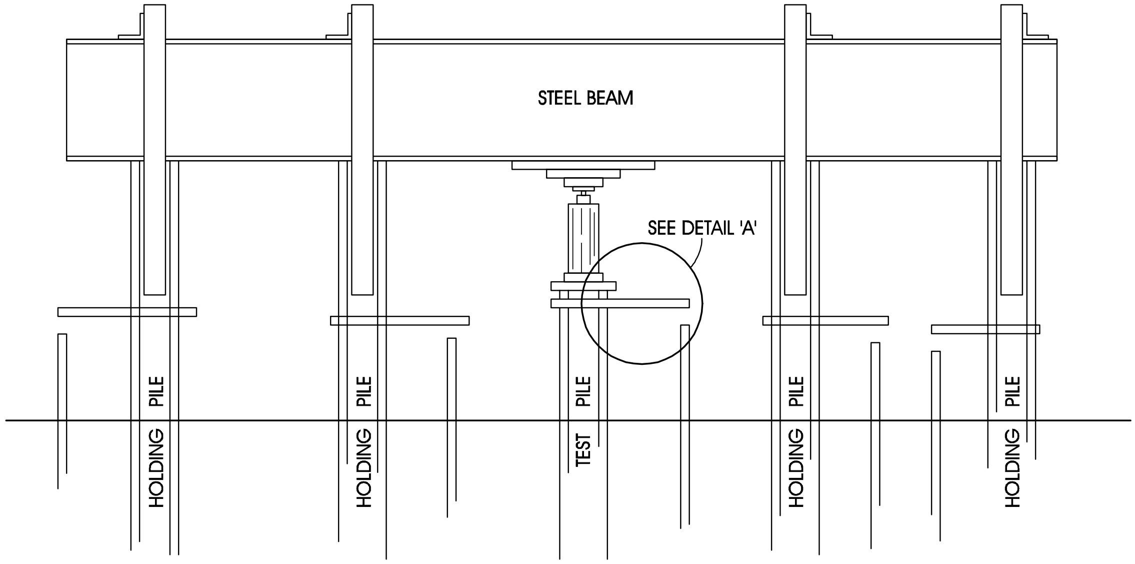
$$\text{Metric: } e = \frac{\text{Load (newtons)} \times \text{Length (meters)}}{\text{Area (m}^2) \times (2.0 \times 10^{11})}$$

Example:
12 x 53 H-Pile
12” (310 mm), 53 lbs (79 kg/m), 65 ft (16 m) long, 30 ton (27.2 metric ton) pile, 60 tons (534 kN) test.

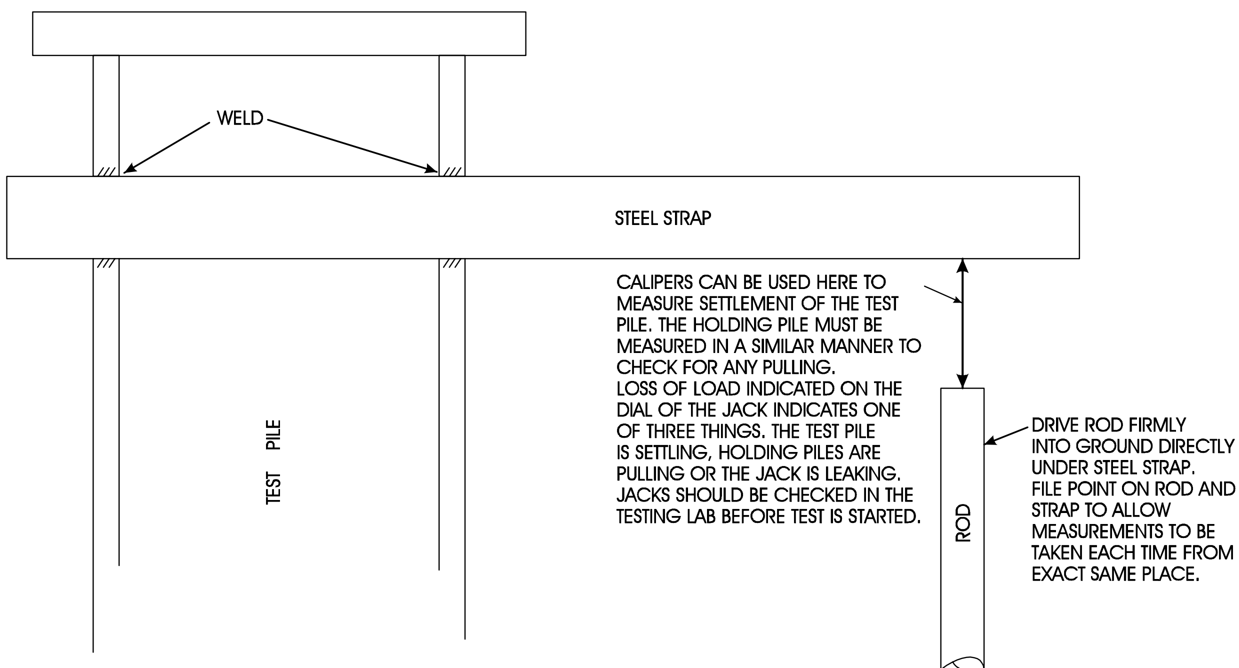
$$\text{English: } e = \frac{[(60\text{ T})(2000\text{ lbs/T})] \times [(65')(12''/\text{ft})]}{(15.6\text{ in}^2) \times (2.9 \times 10^7\text{ lbs/in}^2)} = 0.017\text{ in.}$$

$$\text{Metric: } e = \frac{534\,000 \times 16}{(0.01) \times (2.0 \times 10^{11})} = 0.0043\text{ m} = 4.3\text{ mm}$$

Figures 5-19 and 5-20 on the following pages illustrate pile load tests.



THIS DRAWING SHOWS A COMMON METHOD OF TESTING USED BY MANY CONTRACTORS IN THIS AREA.
THE METHOD OF MEASUREMENT SHOWN IS ONLY SUGGESTED AS ONE OF SEVERAL THAT CAN BE USED.



DETAIL 'A'

Figure 5-19: Sample Pile Load Test

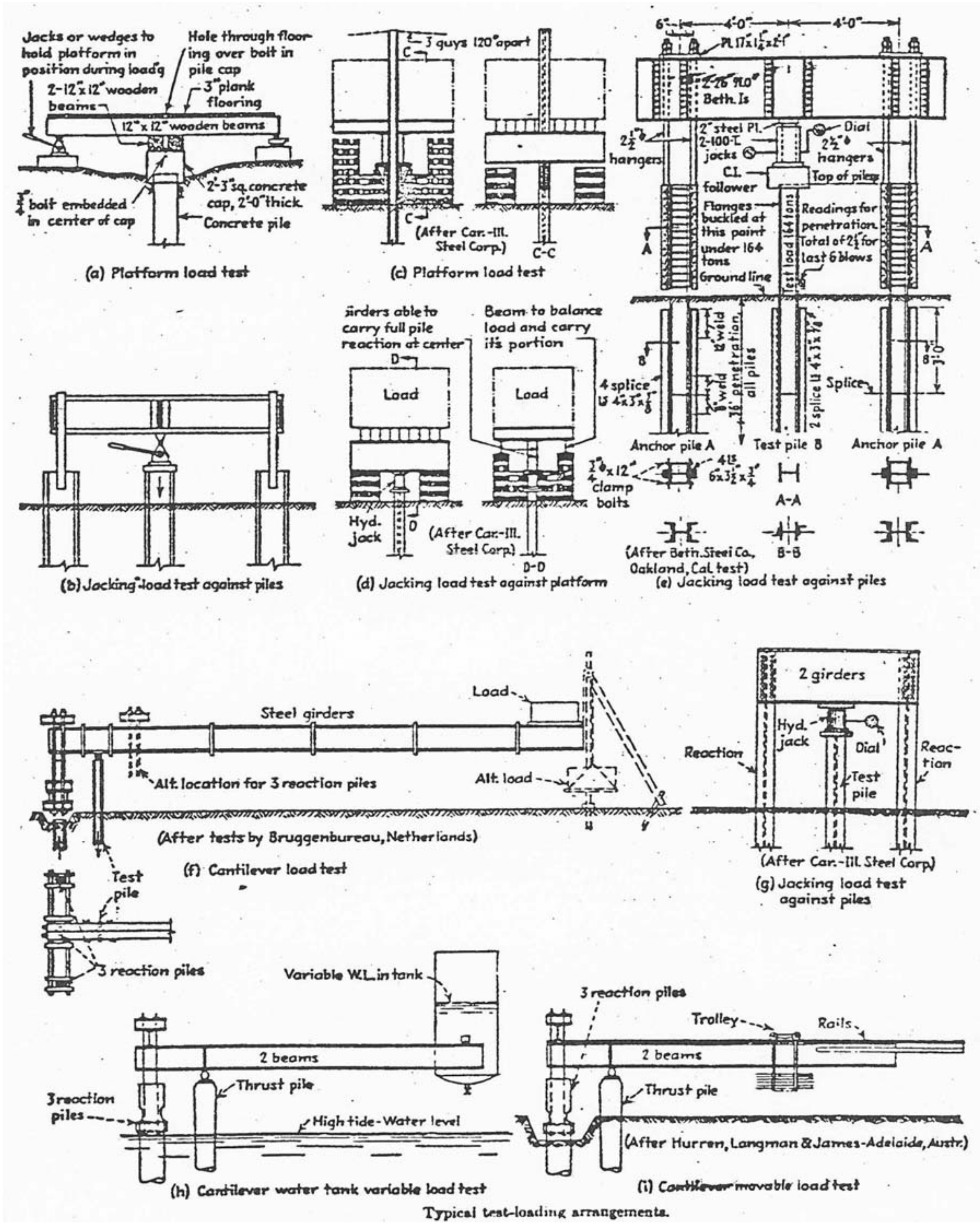


Figure 5-20: Pile Load Tests

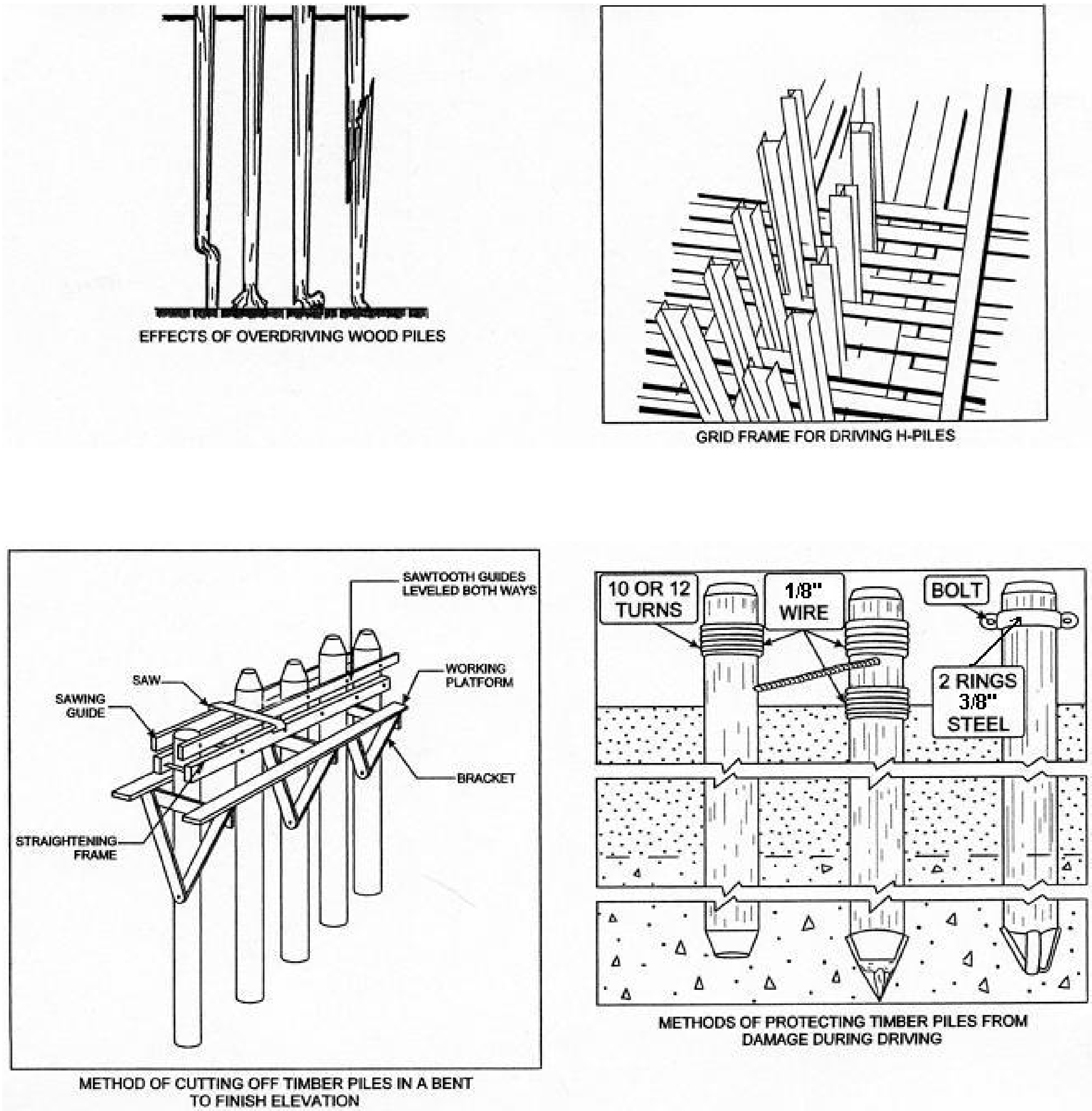


Figure 5-21: Additional Pile Driving Illustrations

SECTION 520 - PORTLAND CEMENT CONCRETE

520.1 - GENERAL

The purpose of this section in the manual is to provide information to supplement the specifications with respect to the item of concrete. The five classes of concrete are designed primarily on the basis of a presumed strength. No matter what class of concrete is involved, the variety of inspection duties that the contract administrator must perform is generally the same. The item of concrete includes forming, placement, finishing, and curing. All aspects must be accomplished properly for successful concrete masonry.

520.2 - MATERIALS

At the present time most all of the concrete used on our Department projects comes in transit mixers from approved concrete batching plants. The specifications are quite complete in detailing requirements of the batch plant and mixers that must be met before they are approved by the Materials and Research Bureau. Therefore, control and inspection of the plant and materials at the plant consists of only the testing and sampling that is done by the contract administrator's representative, the plant inspector. The plant inspector's testing and sampling procedures are detailed under section 520.3, subsections J through Q of this manual. As previously stated, personnel from the Bureau of Materials and Research perform periodic checks of the plants and truck mixers supplying concrete for Department projects. However, the plant inspector should be knowledgeable of the concrete plant specifications for overall understanding of concrete batching.

520.3 - CONSTRUCTION OPERATIONS

A. Review of Plans:

When time permits at the beginning of the project, the contract administrator will find it very helpful to review the plans and make the necessary computations to verify concrete quantities for the record book. Most concrete items are final pay (F) items so time consuming, exact computations of the concrete quantities are not necessary. However, a rough check should be made to assure that there was no gross error (±25 %) made in the calculation of design quantities. The verification of quantities is should be done before payment. When it is done at this time, it familiarizes field personnel with the shapes of the structures, the sizes of pours, and perhaps any typographical errors or inconsistencies in the design plans.

Be sure to check the plans for the class of concrete to be used in different sections of the structure. For example, all abutment backwalls are to be Class AA concrete or the same mix design as the deck, as stated on the plans. If this is not indicated on the plans, the necessary corrections should be made.

Well in advance of grading the bridge seat forms, the contract administrator should check the bearing area elevations with the finished roadway grade by adding depths of materials. Any increase in height plus the camber of the delivered beam sometimes exceeding the design plus part of the plan dead load deflection not being totally realized, all added together, may indicate a negative blocking distance on parts of the span. If so, the contract administrator should discuss this problem with the District Construction Engineer to see if the bridge seats can be lowered slightly, thus eliminating a hump in the deck, curb, sidewalk and guardrail.

B. Materials and Research Bureau:

As soon as the contractor has designated the concrete supplier, the contract administrator should contact the Materials and Research Bureau in Concord and request verification that design mixes have been submitted by the supplier and have been approved for each class of concrete required for the project. Even though the contract administrator may know that the concrete lab has been informed by the supplier, this should be verified by personally requesting the design mixes and leaving his/her home mailing address for future correspondence. There is a sample Concrete Mix Design form on page.

C. Design Mix:

The design mix received from the lab will typically provide concrete acceptable for all uses on the project based upon historical data. However, there are times when workability or the advantages of admixtures may require field adjustment of the mix for specific pours. Should the concrete mix design be repeatedly unsatisfactory, the contract administrator should have the supplier redesign and resubmit the mix after a complete evaluation rather than continue with field adjustments. Adjusting the water and aggregate in the mix will only be done after testing in the field proves the designed mix is not meeting requirements. The factors determining these adjustments will be discussed in the field testing section.

The proper use and dosage of admixtures such as air-entraining, water-reducing, and retarding admixtures is generally determined from previous experiences. The initial starting dosages are recommended by the supplier for the intended use and may need adjustment once the concrete is tested. If there is any question as to their uses and the quantities to add, the District Construction Engineer or other field personnel with experience in this area should be consulted.

Ordinarily, using the air meter field test will disclose whether or not enough air entraining admixture is being incorporated into the mix. Adjusting the amount up or down by fractions of an ounce (ml) per cubic yard (cubic meter) can usually be called in to the plant over the transit mix truck's radio or by telephone until further testing of adjusted mixes shows the proper air content. Redosing the air entraining agent in the field will enable a load with low air content to be used, but this responsibility should always be restricted to qualified quality control personnel from the supplier. The contract administrator or NHDOT personnel should not make dosage recommendations, only check for compliance.

Regarding the use of retarder, it takes experience to evaluate factors such as: the weather; the time it will take to place the quantity of concrete; the concrete placing performance of the contractor, including his/her ability to finish the concrete once placed. When in doubt as to the amount of retarder with which to start a pour and the stages at which this amount should be cut down, the District Construction Engineer and/or other field personnel should be consulted. Figure 5-22 shows a graph depicting the relationship between temperature and time to set the concrete when using retarder.

Approved concrete admixtures are on the Materials and Research Qualified Products List at http://www.nh.gov/dot/materialsandresearch/pdf/apl.pdf.

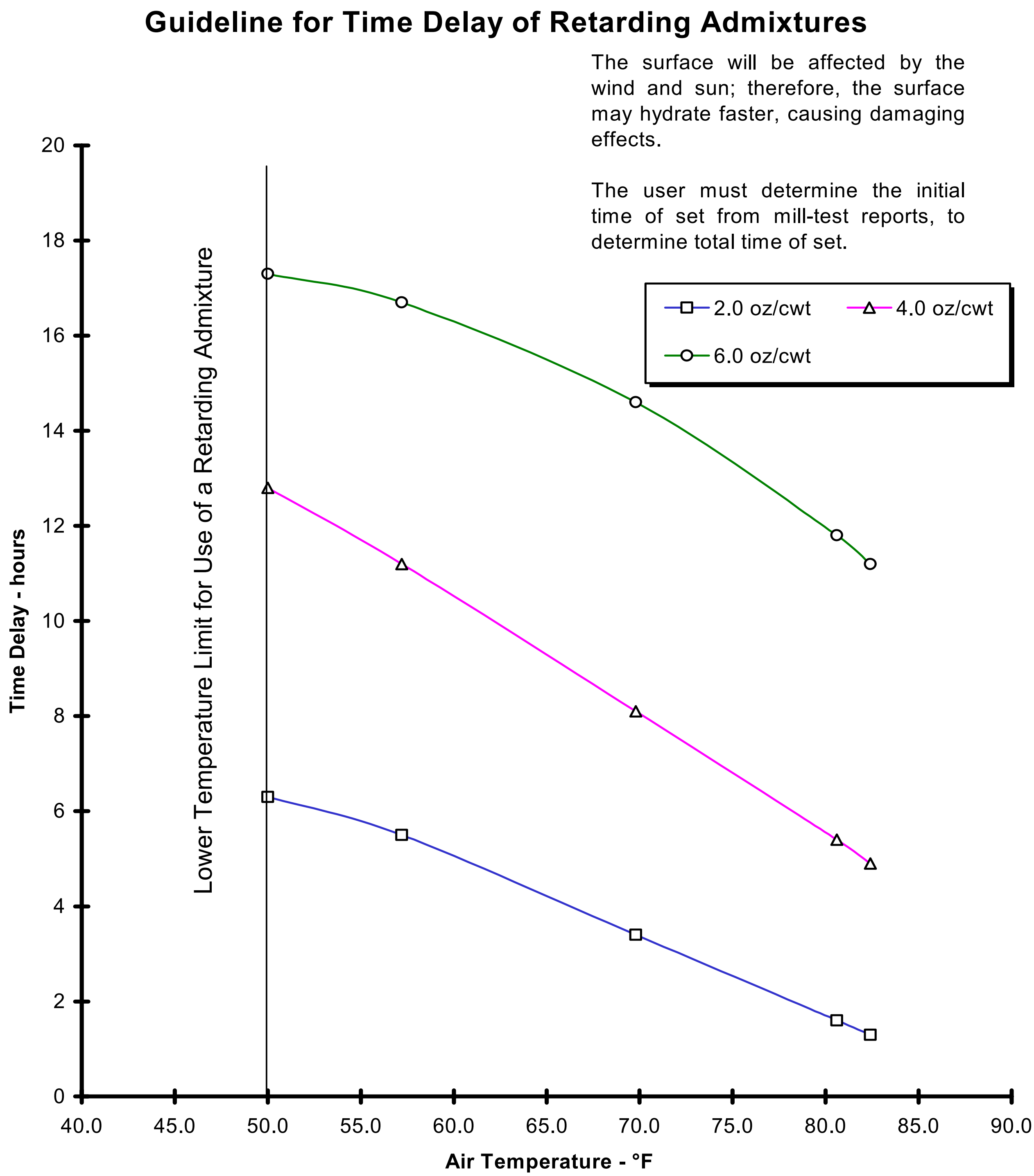


Figure 5-22: Effects of Retarding Admixture on Concrete Setting Time

D. Subgrade Preparation:

Subgrades should be graded to their specified elevations and should be moist when the concrete is placed. A moist subgrade is especially important to prevent rapid extraction of water from the concrete when footing or slabs are being placed. Where the foundation is rock, all loose material should be removed before the concrete is placed. Pressure washing or the use of compressed air are possible recommended methods of cleaning. In general, when it is necessary to cut out ledge or rock the surfaces should be vertical or horizontal (or in combination), not sloping.

E. Checking Forms:

The specifications are quite detailed in discussing formwork. Forms should be clean, tight, adequately braced, and constructed of materials that will impart the desired texture to the finished concrete. Plywood panels should be assembled with the grain running horizontally. Care should be taken to see that sawdust, nails and other debris are removed from the sections to be placed. Usually, during the excavation operations or preparation of the subgrade, batter boards are constructed to locate the centerlines of bearing and roadway. The contract administrator must be confident that these off sets are correctly located, and use these points to check the assembly and verify the location of the forms. The forms must be set vertical, horizontal or battered as required and securely braced to stay true. Trueness should be continually monitored throughout the placement with the use of plumb lines or other accepted methods. After the form is completed by the carpenters, the contract administrator should check it to ensure that it is of the proper size and will be strong enough to resist deformation under the weight of the concrete. In addition to line, the contract administrator will verify the elevation of the top of the pour at several different locations on the form. The bench marks should be the same, relative to each other, and should be checked against each other periodically. In addition to checking this elevation against several bench marks on the job, it is advisable to set up a level immediately after the pour and check the top of the concrete elevation again. This checking procedure is a must when pouring bearing areas with precise finished elevations. Finally, when the forms and resteel are complete, the clearance between the steel and the outside of the form and top of the concrete must be checked. It is important to have the proper clearances as shown on the plans. Using templates or string lines to illustrate the finished concrete grade will provide a method to check clearance of top mats in sidewalks and walls.

F. Stone Masonry Formwork and Chamfer Strips:

Two items of particular interest with regard to formwork have been set aside for emphasis in this subsection.

1. Stone Masonry Formwork:

   There is a special consideration when constructing forms intended to hold concrete against stone masonry walls. Usually, the face of the wall has been built using irregular quarry or field stone behind which the concrete will be placed. Naturally, support to each individual stone must be provided by some sort of formwork so that the weight of the plastic concrete being placed against them will not dislodge the stones and cause a complete collapse. Figure 5-23 below shows an ideal approach to forming and supporting a concrete wall incorporating a stone masonry face. However, few contractors consider this application economically feasible and, therefore, use some combination of walers and wedges in lieu of a sand cushion.

Each stone usually requires at least two wedges. This method is depicted in Figure 5-24 below.

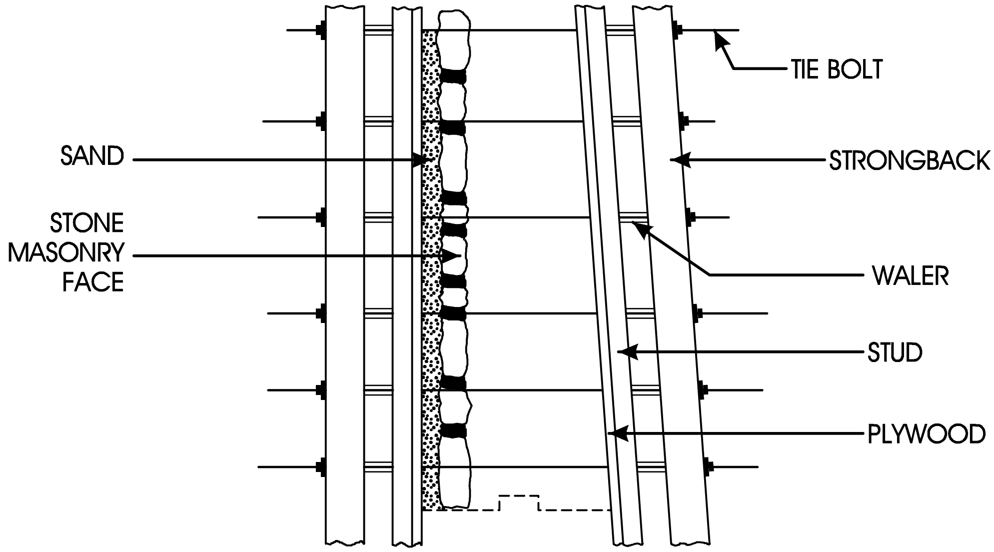


Figure 5-23: Sand Method of Securing Stone Masonry Faces for Concrete Placement

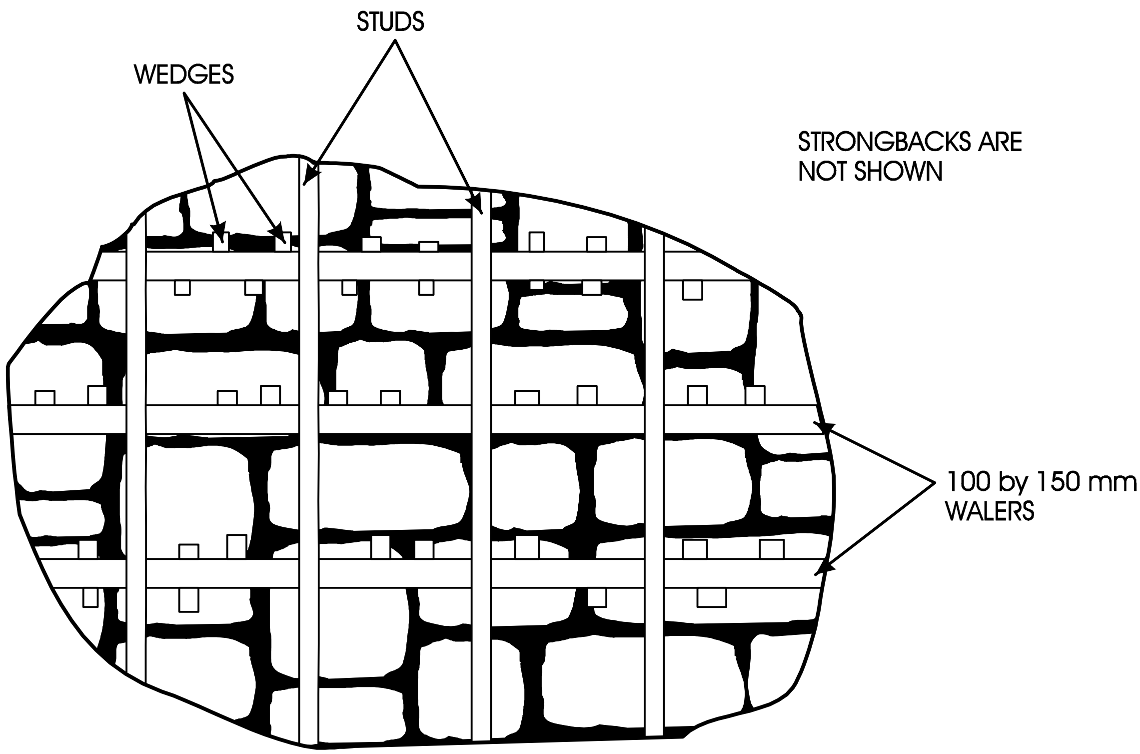


Figure 5-24: Wedge Method of Securing Stone Masonry Faces for Concrete Placement

Alternatively, some designers or contractors opt to pour the concrete first, allowing for the stone facing to be constructed and secured to the concrete afterwards through the use of dovetail anchors that are inserted and twisted into slotted strips embedded vertically in the concrete.

2. Chamfer Strips:

   The use of chamfer strips at the exposed corners and edges of finished work is generally required to provide an eye-appealing line on the concrete. Naturally, when shown on the plans, they should be used. Almost without exception, chamfer strips should be used on the front three sides of bridge seats even though forms are sometimes cut to grade. Their use here protects the edges from breaking or chipping. The District Construction Engineer should be consulted if there is any doubt as to the use of chamfer strips on the project.

G. Falsework:

Falsework is the term that describes the load bearing portions of forms where the form supports the concrete vertically rather than from the side. Cantilevered portions of pier caps, decks, abutments and wings, and slabs of rigid frame bridges and culverts are some of the placements that require careful consideration in the design of falsework. The falsework must be sufficient to withstand the placement of concrete loads without deformation. Falsework plan submissions to the Construction office are required of the contractor three weeks prior to the beginning of concrete operations so that there is sufficient time for review and documentation. In this manner, the contract administrator can inspect construction of the falsework forms for compliance with the accepted plan. Falsework on decks usually necessitates timber being used around the steel girders and the abutment. The contract administrator must inspect this area to ensure that no timber will bind between the steel and the backwall should the bridge expand due to change in temperature. Falsework timber should be inspected for strength deficiencies such as cross grain, large knots, cracks, and dry rot.

The following are recommendations for submitting bridge falsework plans by the contractor for documentation:

1. Five sets of all falsework plans and calculations shall be submitted to the Construction Bureau directly or through the contract administrator.

2. The specifications require that the contractor’s falsework plans be stamped by a New Hampshire Registered Professional Engineer.

3. All falsework drawings shall be to a suitable scale, neatly drawn, and all details shall be clearly shown. Especially for lumber the species, grade, and allowable basic stresses in bending, horizontal shear, compression perpendicular and parallel to grain, and the modulus of elasticity shall be noted on the submitted plans. Unless indicated otherwise, the stock shall be assumed as dressed lumber.

4. For construction purposes, the basic allowable stresses shall be increased by 25%.

5. Computation for deflection shall be made in order to have the resulting shape match the design with satisfactory tolerance. Live load may be omitted from wood falsework deflection computations. For spans of 12 feet or less, wood falsework shall have a resulting deflection of less than one six hundredth (1/600) of the span.

6. Falsework resting on the original ground or on compacted fill shall be supported by continuous sills that are suitably spliced. For wood falsework, the soil shall not be expected to support more than 2000 lbs/ft^2.

7. Special attention shall be given to the falsework's lateral bracing in all planes of movement.
8. The use of shimming devices, such as hardwood wedges, for vertical control of settlement shall be clearly shown.
9. Special proprietary devices such as jacks, hangers, and steel scaffolding shall be referenced to a standard catalog, or a catalog may be supplied with the submission. Such catalogs shall supply all the required details and the allowable loading recommended for the device in question.
10. Any exterior beam overhang supports (brackets) that induce excessive torsion into this beam shall be prohibited.

H. Expediting Delivery:

In talking to the contractor, the contract administrator should discuss aspects of delivering and placing the concrete before it arrives on the job to ensure that the work area is properly organized. Access for the mixer trucks should be given consideration, and chutes and tremies must be properly arranged for an efficient procedure. At this time, the contractor should assure the contract administrator that the necessary personnel are available with standby vibrators and generators are on hand should there be a breakdown. The contractor should be advised that aluminum chutes, tremies, and finishing tools are prohibited.

I. Weather Conditions:

Rainy weather is another variable that the contract administrator has to contend with once the forms are up, checked, and the contractor is ready for the placement. Concrete placements should never be started in the rain unless adequate protection can be provided. In addition to rainwater being added to the concrete and affecting the water-cement ratio, rain spots will appear on surfaces that are uncovered. Concrete shall not be placed when the atmospheric temperature is below 35°F without provisions for external heat with housing or insulated forms and blankets.

J. – Q.: Comprehensive Guide for Concrete Plant Inspectors

The following sections ("J" through "Q") provide a comprehensive guide for use by **concrete plant inspectors**. These sections review concrete plant testing procedures, record keeping, and general responsibilities and duties expected of a concrete plant inspector.

J. Concrete Plant Inspectors:

The concrete plant inspector is the contract administrator's representative at the batching plant and he/she inspects the preparation of the concrete mix. The plant inspector may be one of the assigned project personnel, an inspector from another project, or a Department hired consultant inspector. Should the contract administrator need a plant inspector to cover a placement aside from the personnel assigned to the project, arrangements will need to be made with the current low bidder consultant testing company as directed by Materials and Research. These inspection companies are generally short staffed and very busy, make sure to give them as much lead time as is possible. The information required on the concrete inspector's work sheet (also known as the "call" sheet) should be filled out in full and relayed to the consultant's office when requesting coverage.

K. Concrete Plant Testing:

Before a concrete plant is approved to be used in state work, the NHDOT concrete laboratory personnel (Materials and Research Bureau) have made in depth, on-site inspections. The plant inspector has many spot checks to perform in order to provide consistent quality concrete. Upon receiving the concrete plant work sheet (call sheet), the inspector should review the information for clarity and completeness. Now is the time to call with any questions. If need be, call the contact person at the “HOME” telephone number listed on the concrete plant work sheet (call sheet). Waiting to call until that person is on site may delay the entire placement.

The plant inspector must arrive at the plant in sufficient time to perform tests and inspections before the first load is ready to be batched out. The time anticipated for the first truck to arrive on site is indicated on the concrete plant work sheet (call sheet). This time is critical and should not have to be pushed back due to delays in plant inspection. Plan accordingly.

After introducing himself/herself to the plant batch person, the inspector should ask if there is sufficient cement and materials on hand to cover the placement. Double check that the batch person’s information and the concrete plant work sheet (call sheet) are the same. See if you are “on the same page”. You may be asked to sign in on the plant’s logbook. If such a book exists remember to post a copy of your test results inside before leaving for the day.

Next, the plant inspector should gather aggregate samples for the gradation and fineness modulus (FM) tests. Moisture content calculations will be used to compute the actual concrete mix by adjusting the design weights shown on the concrete plant work sheet (call sheet). An illustration of these calculations follows:

L. Concrete Plant Moisture Content Calculations:

There are several theoretical factors that must be related to moisture content testing. Batch weights used in the design of any concrete mixture are based upon saturated surface dry (SSD) aggregate weights. However, equipment required at each concrete plant, by specification, and the drying procedure used cannot compare the actual sample with the SSD standard. Therefore, a correction factor known as the absorption factor must be used. This factor is included in the mix design provided by the Bureau of Materials and Research prior to the start of non-QC/QA concrete work on any project. The information will be forwarded to the plant inspector on the concrete plant work sheet (call sheet). If the absorption factor is unknown or unavailable, use 1% and make note of your assumption on the field test report.

To perform the tests the inspector should choose a representative sample from the bins or stockpiles. Since much of the excess water will sink to the bottom of the storage bins during the night, if the plant has not been operating prior to your arrival, have the batch person drop some aggregate before taking your sample. Prior to taking a sample from a stockpile, have the plant personnel mix the pile and build a sample platform. Obtaining a representative sample is extremely important to the outcome of your results. Do not take a sample until you feel you will be able to get a representative one. Once batching operations begin, ensure that the material being fed to the plant is of the same nature as the material you sampled.

Damp aggregates will normally have about 3% moisture in the sand and 1% in the stone. Very wet sand may have 6% to 10% moisture and the stone may hold 3% to 4%.

Therefore, it is very important to know how much free water must be compensated for in order not to exceed the proper water/cement ratio.

Weigh out about 300 grams, or about ¾ of a pound of damp sand containing the unknown moisture content. Record the "wet weight". Dry the sample on the stove or heating device provided at the plant laboratory, and record the "dry weight". Figure the moisture content following the example below:

Weight of wet sand and pan	345 grams
Minus tare weight of the pan	-45 grams
Wet Weight of sand	**300 grams**
Weight of dry sand and pan	332 grams
Minus tare weight of the pan	-45 grams
Dry Weight of sand	**287 grams**

300g – 287g = **13grams** of water

$$\textbf{Moisture Content} = \frac{\text{13 grams water}}{\text{287 grams of dry sand}} = 0.0453 \Rightarrow \mathbf{4.53\%}$$

The sample lost 13 grams and is now absolutely dry. The mix design gives SSD values for each aggregate and the absorption factors.

M. Other Concrete Plant Testing Procedures and Calculations:

Let's say a mix design calls for 1130 pounds of sand with an absorption factor of 0.6%. An absorption factor of 0.6% means that the aggregate will soak up a weight of water equal to 0.6% of its dry weight. The portion of the moisture content that remains once the particles are saturated will give an indication of the "free water" in the material. This free water is the water available in the aggregate stockpile for hydration with the cement.

Free Water = 4.53 % - 0.6% = 3.93 % => **use 4%**

Looking at our example, if 1130 pounds (512 kg) of sand is weighed out there will actually be 1130 x 0.04 = 45 pounds (512 x 0.04 = 20 kg) of free water contained in that sample. You only have 1130 – 45 = 1085 pounds (492 kg) of sand. The mix design says you need 1130 pounds (512 kg) of sand. While not an absolutely accurate method, it is close enough to add an extra 45 pounds (20 kg) of sand to the mixture to account for this difference.

The procedure for testing and calculation of the moisture content of the coarse aggregate is similar. The absorption correction factor is likely different and would need to be used.

The amount of water to be added at the plant is not stated on the concrete plant work sheet (call sheet) for a reason. The plant inspector and the contract administrator should decline to tell concrete company personnel the amount of water that is to be added to the mix. The inspector should inform the batch person of the moisture content of aggregates from the tests, the slump to be used at the project, and have the batch person decide how much water to add. The batch person knows the theoretical water for each yard and can estimate the amount to hold back for mixing. Write the amount added to the load on the delivery slip. At the project, the transit mix driver may add water to provide for the specified slump and field personnel should note this amount on the slip. By not specifying the amount of water in gallons to be added to the mix, the State inspector cannot be made to share in responsibility for wet loads. The water amounts noted on the delivery slips (the aggregate free water, the water added at the plant, and the water added on the site) can be used to figure the water cement ratio.

Testing proceeds by checking the fineness modulus of sand, which should be per specifications between 2.5 and 3.1. The F.M. used in designing the mix will be shown on the concrete mix design sheet provided from Materials and Research. If the F.M. varies more than 0.2+/- from that used in the design, a redesign of the mix may be necessary. The test used for determining the fineness modulus may be found on the following page.

The coarse aggregates should be run through the sieves for gradations. To help the concrete plant inspector in recording information on the field test reports and transit mix delivery slips, the concrete plant work sheet (call sheet) should be used. All tests must be completed prior to the batching of the first truckload. A completed gradation test report should be delivered to the on site project personnel with the first delivery slip. A completed gradation test report should be delivered to the on site project personnel with the first delivery slip.

Testing procedures for fine and coarse aggregate used in concrete; and samples of concrete plant work sheets, gradation test reports and concrete batch and delivery records are shown on the next 10 pages. The first 2 pages give the testing procedures. The following 4 pages are of worksheets, a test report, and delivery slip for a sample project done in English units (Figures 5-25 thru 5-27). The last 4 pages are of a sample project done in metric units (Figures 5-28 thru 5-30).

N. Test of Fine Aggregate Used in Portland Cement Concrete:

1. Scope: Fine aggregate used in the production of Portland cement concrete has a definite effect on the design of a concrete mix. Besides influencing the amount of mixing water required to make a workable mix, the mechanical composition of the aggregate is of concern since it affects the strength of the mix. As a result, engineers use an empirical index called the fineness modulus (FM) as a measure of the fineness of an aggregate. In general terms, a fine aggregate with a high FM is coarser than one with a lower FM.

The purpose of this test is to determine the fineness modulus, (FM) of the fine aggregates used in Portland cement concrete.

2. Apparatus:
   - A representative sample of the fine aggregate.
   - A scale sensitive to 0.1 grams.
   - Eight nested sieves as follows: 3/8”, #4, #8, #16, #30, #50, #100, and #200 (9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm, 0.600 mm, 0.300 mm, 0.150 mm and 0.075 mm) and a pan.
   - Mechanical shaker.

3. Procedure:
   - Dry about 700 grams of fine aggregate.
   - Arrange the sieves in descending order so that the 3/8” (9.5 mm) sieve is on top, the #4 (4.75 mm) sieve is next and so forth down to the #200 (0.075 mm) sieve, which should be resting on the pan.
   - When the fine aggregate has cooled, weigh out a sample of approximately 500g of material.

- After recording the weight, place the sample of material in the 3/8” (9.5 mm) sieve, cover, and clamp the entire nest of sieves into the mechanical shaker.
- Shake for 3 to 5 minutes.
- Remove the 3/8” (9.5 mm) sieve and weigh the material retained on it. Record this weight.
- Next add the material retained on the #4 (4.75 mm) sieve to the material previously weighed and record this cumulative weight. Continue this process for the remaining sieves, recording the cumulative weight for each sieve.
- Calculate the percent of material retained for each sieve by using the cumulative weight recorded, divided by the original known weight of the sample.
- The FM is calculated by dividing the summation of percents retained on all sieves except the #200 (0.075 mm) sieve by 100. Expressed mathematically the formula is:

$$FM = \frac{\text{SUM OF \% RETAINED (not including the \#200 sieve)}}{100}$$

4. NOTE: The determination of the percent passing, required for filling out the gradation test report (otherwise known as the “gray sheet”), is found by simply subtracting the percent retained from 100.

O. Test of Coarse Aggregate Used in Portland Cement Concrete:

1. Scope: In addition to the fine aggregate test described above, the concrete plant inspector is also required to run a gradation on the coarse aggregate. The procedure is similar to that used to determine the gradation of the fine aggregate.

2. Apparatus:
   - A representative sample of the coarse aggregate.
   - A scale sensitive to 0.01 pounds (0.005 kg).
   - The following sieves: 1 ½”, 1”, ¾”, ½”, 3/8”, #4, #8, #16,and #50 (37.5 mm, 25.0 mm, 19.0 mm, 12.5 mm, 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm and 0.300 mm). The #16 and #50 (1.18 mm, 0.300 mm) sieves are only used for an overlay concrete mix design.
   - Mechanical shaker.

3. Procedure:
   - The gradation test of the coarse aggregate is run similar to that of the fine aggregate (see previous page). Calculate the percent passing for each sieve that is required for your coarse aggregate sample according to Table 3, Section 520.2.2.3.2 of the Standard Specifications. Where the storage of the coarse aggregate is in two or more bins, a sample for each bin is taken and a gradation test is performed on the individual sample. It is then necessary to combine the results of the individual tests so a comparison with the specification can be made.
   - In combining the test results, it is necessary to know how the aggregates are proportioned in the concrete mix design. The percentage of each size of coarse aggregate can be determined from the plant inspector’s work sheet (call sheet). For instance, the example concrete plant work sheet (call sheet) on the following

pages shows approximately 60% of #4-3/8” stone (106.9 ÷1780) and 40% of 3/8” – ¾” stone (711÷1780).

- In this case, multiply the gradation test result for the #4-3/8” (4.75 to 9.50 mm) stone by 0.60 and the result for the 3/8”-3/4” (9.5 to 19.0 mm) stone by 0.40. Add these results, which represent the proportioned gradation, and compare this value with the specifications. If in doubt as to the proportioning, consult the concrete mix design sheet from the Bureau of Materials and Research and use the corresponding percents as the multiplier.
- See the following pages for an example of documentation for this test procedure.

Figure 5-25: Concrete Plant Work Sheet (Call Sheet) - English

(Rev. 3/96)

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION

BUREAU OF CONSTRUCTION
CONCRETE PLANT WORK SHEET (ENGLISH)
(Also referred to as the call sheet)

Project Keene 12345
Contract Administrator Ronald Tanner
Plant Location Whitcomb – Keene
Bridge Location 082/130 Route 12 over the B&M RR
Section being poured Abut A backwall
Absorption Factor (fine aggregate) 0.746%
Absorption Factor (coarse aggregate) 0.438%
No. of bag mix 7
Heat Aggregate ☐ Cool Aggregate ☐ Hot Water ☒

Date March 19, 1996
Project Phone 669-5554
Pager Number 662-0132
Home Phone 946-7381
Cell Phone 419-1178
Concrete Class AA Item No. 520.01
Estimated CY 30
Design F.M. 2.65
Slump 2-4 inches
Water/Cement 0.444
Pour start time 7 AM on site

A	B	C	D	E	F	G	H	I
Aggregate Size	1 CY dry weight (LBS)	% free water minus absorption factor (÷100)	Gallons free water per CY C x B / 8.33 =D	1 CY wet: (LBS) (C x B) + B = E	(8) CY wet E x (8) = F	Gallons free water (8) CY D x (8) = G	(3) drop weights F / (3) = H	Accumulated drop weights (if used) (LBS))
Sand	1185	0.036	5.12	1227.66	9821.3	41.0	3273.8	3273.8
#4- 3/8”	1069	0.01	1.28	1079.69	8637.5	10.2	2879.2	6153.0
3/8” – 3/4”	711	0.005	3.56	714.56	5716.5	28.5	1905.5	8058.5
Cement	329	lbs/CY		x ( ) CY =	2632		877	
Pozzolan	329	lbs/CY		x ( ) CY =	2632		877	
Air Ent.	2.6	oz/CY		x ( ) CY =	20.8			
Midrange	30	oz/100 lb	x (C+P)/100 x ( ) CY =		1579.2			
HRWR		oz/100 lb	x (C+P)/100 x ( ) CY =					

A	F	G	H	I	F	G	H	I
Aggregate Size	( ) CY wet E x ( ) = F	Gallons free water ( ) CY D x ( ) = G	( ) drop weights F/( ) = H	Accumulated drop weights (if used) (LBS)	( ) CY wet E x ( ) = F	Gallons free water ( ) CY D x ( ) = G	( ) drop weights F/( ) = H	Accumulated drop weights (if used) (LBS)
Sand								
#16 – 3/8”								
3/8” – 3/4”								
Cement								
Pozzolan								
Air Ent.								
Midrange								
HRWR								

The information within the dotted lines above is to be provided by the contract administrator. The remainder of the worksheet is to be filled out by the plant inspector.

3/8"	Weight (LBS)	Tare (LBS)	Net Weight (LBS)				
Total Weight	15.40	1.10	14.30	**% Retained**	**% Passing**	**% Stone Mix**	**%**
½"	1.1	1.1	0	0	100	60	60
3/8"	7.15	1.1	6.05.	42.3	57.7	60	34.6
#4	14.0	1.1	12.90	90.2	9.8	60	5.9
#8	14.97	1.1	13.87	97.0	3.0	60	1.8
#16	15.06	1.1	13.96	97.6	2.4	60	1.4
#50	15.13	1.1	14.03.	98.1	1.9	60	1.1

¾"	Weight (LBS)	Tare (LS)	Net Weight (LBS)				
Total Weight	26.45	1.1	25.35	**% Retained**	**% Passing**	**% Stone Mix**	**%**
1"	1.1	1.1	0	0	100	40	40
¾"	7.41	1.1	6.31	24.9	75.1	40	30.0
3/8"	18.92	1.1	17.82	70.3	29.7	40	11.9
#4	24.27	1.1	23.17	91.4	8.6	40	3.4
#8	26.15	1.1	25.05	98.8	1.2	40	0.5

$1^1/_2$"	Weight	Tare	Net Weight				
Total Weight				**% Retained**	**% Passing**	**% Stone Mix**	**%**
2"							
$1^1/_2$"							
¾"							
3/8"							
#4							

	3/8"	3/4"	$1^1/_2$"	Total	Spec
2"					
$1^1/_2$"					
1"	60	40		100	100
¾"	60	30		90	90-100
3/8"	34.6	11.9		46.5	20-55
#4	5.9	3.4		9.3	0-10
#8	1.8	0.5		2.3	0-5

Moisture

	Weight	Tare	Net Weight	
Wet Sand	474.5	20	454.5	**A**
Dry Sand	455.6	20	435.6	**B**
	Wet - Dry		18.9	**C**

$\frac{C}{B} = \frac{18.9}{435.6} = 4.34\ \% - 0.746\% = 3.6\ \%$

(Absorption)

SAND	Weight (g)	Tare (g)	Net Weight (g)			
Total Weight	455.6	20	435.6	**% Retained**	**% Passing**	**Spec**
#4	24.4	20	4.4	1.0	99.0	95-100
#8	74.4	20	54.0	12.4	87.6	
#16	148.5	20	128.5	29.5	70.5	45-80
#30	250.0	20	230.0	528	47.2	
#50	355.4	20	335.4	77.0	23.0	10-30
#100	430.8	20	410.8	94.3	5.7	2-10
			F.M. →	2.67		
#200		20		97.8	2.2	0-3

Tested By Michael M. Peters

Figure 5-26: Concrete Gradation Test Report - English

(Rev 5/2000)

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION
GRADATION TEST REPORT (ENGLISH)

Project: Keene 12345 | Field Test No.: MMP031996-01
Type of Material: Concrete Aggregate | Date Reported: March 19, 1996
Reported by: Michael M. Peters | Received by Lab: ____
Report to: Project Files ☒ Lab ☐ Contractor ____ ☐
Sampled: March 19, 1996 At (town): Keene, NH
Source of Material: Whitcomb Materials
Sample From: Stockpiles Pit ☐ Roadway ☐ Sta ____
Quantity (Represented or Estimate): 30 CY +/-
Purpose / Location: Abut A Backwall Item No. 520.01
Tested for: Gradation ☒ F.M. ☒ % Moisture ☒ Date: March 19, 1996

Sieve	Size 3/8 60 % % Passing	Size 3/4 40 % % Passing	Size % % Passing	Combined Results	Required Spec.
	Coarse Aggregates and Gravels				
6”					
$3^1/_2$”					
3”					
$2^1/_2$”					
2”					
$1^1/_2$”					
$1^1/_4$”					
1”	60.0	40.0		90.0	100
3/4”	60.0	30.0		90.0	90 – 100
1/2”					
3/8”	34.6	11.9		46.4	20 – 55
#4	5.9	3.4		9.3	0 – 10
#8	1.8	0.5		2.3	0 - 5
#16					
#50					
#200 in Total					
% Fract’d Faces					
	Fine Aggregates and Sands (Washed ☐)				
#4	99.0				95 - 100
#8	87.6				
#16	70.5				45 – 80
#30	47.2				
#50	23.0				10 – 30
#100	5.7				2 – 10
F.M.	2.67				2.65±0.2
#200 in Sand	2.2				0 - 3
% Moisture	3.6				

Remarks: ____

Meets requirements for: Item 520.01 Class AA Concrete See reverse ☐

Tested by: Michael M. Peters
Signature

When the inspector has completed the above gradations, the following form needs to be filled out and sent to the project on the first load along with this batch and delivery record. This is the placing inspector's opportunity to verify that all the aggregate gradations meet specifications. One copy of each gradation test shall be given to the contract administrator for his/her records and also be maintained at the concrete plant in a ringed binder as a record of the gradation history.

This side to be filled out completely by the **Plant Inspector** for the first load of each truck size or when the mix changes due to moisture content variations, etc. However, only the top portion needs to be completed thereafter.

Time and amount of water are to be completed on every slip by the **Concrete Placing Inspector**. Other portions are to be completed at appropriate intervals.

(Revised 3/96)

NHDOT CONSTRUCTION BUREAU
CONCRETE BATCH & DELIVERY RECORD
(ENGLISH)

Project Keene 12345 Date 03/19/96
Truck # 23 Batch # 1 Size (A) 8 Sub-Tot 8 CY
Mix Weights on Batch # 1 Total Delivered Water (B) 244 Gal
Total Delivered Weight (C) 30,795 lbs Batch Time 6:25 AM
Michael M. Peters — Plant Inspector; 72665 — Employee Number

Complete the following for the first load of each truck size or when the mix changes
Source Whitcomb – Keene Class AA
Location of pour Abut A Backwall
Scale inspected date Feb 27, 1996 Water Temperature 120°F

	Moisture		Free Water		Batch Weights	
Sand	3.6	%	41	Gals	9820	lbs
#4-3/8"	1.0	%	8	Gals	8635	lbs
3/8" - 3/4"	0.5	%	29	Gals	5710	lbs
#4 - 1-1/2"		%		Gals		lbs
Water added at Plant			164	gallons x 8.33 =	1366	lbs
Total Delivered Water			244	gals (B)		

Cement Wt. 2632 + Pozzolan 2632 = 5264 (G)
Total Delivered Weight (C) 30,795 lbs

	Kind		at plant		
Air Entrainment	Kind	Darex II	at plant	21	oz
Midrange	Kind	WRDA 65	at plant	1579	oz
HRWR	Kind		at plant		oz
	Kind		at plant		oz

Weather Fair Air Temperature 42 °F

Job Arrival 7:05 AM
Start Mixing 7:12 AM R.P.M. 17
Start Discharge 7:18 AM
Finish Discharge 7:42 AM

Water added at site 20 gals x 8.33 = 167 lbs (D)
Total Delivered (B) 244 gals
Total Water (F) 264 gals / size (A) 8 = 29.25 gal/CY

Concrete Temperature 61 °F Slump 2¾" Air 6.2 %
Cylinder Numbers 040 – 042
Unit Weight:
Weight 43.35
- Bucket 7.45 Conc Wt 35.90 / Volume 0.25 = 143.60 lbs/CF (E)
Conc Wt. 35.90

Yield: (C + D) / (E x A) = (30,795+167) / (143.60 x 8) = 27.10 CF/CY

W/C Ratio: (F x 8.33) / (G) = 2199 / 5264 = 0.418 microwave: 0.423

Ronald Tanner — Concrete Placing Inspector; 56627 — Employee Number

Figure 5-27: Concrete Batch and Delivery Slip - English

Figure 5-28: Concrete Plant Work Sheet (Call Sheet) - Metric

(Rev. 4/03)

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION

BUREAU OF CONSTRUCTION
CONCRETE PLANT WORK SHEET (SI)
(Also referred to as the call sheet)

Project	Laconia	Date	8/12/96
Contract Administrator	Ronald Tanner	Project Phone	669-5554
		Pager Number	662-0132
Plant Location	F&S Laconia	Home Phone	946-7381
Bridge Location	082/130	Concrete Class	AA Item No. 520.01
Section being poured	NE Backwall	Estimated m^3	25
Absorption Factor (fine aggregate)	0.746%	Design FM	2.65
Absorption Factor (coarse aggregate)	0.438%	Slump	25-75 mm
No. of bag mix	9.2	Water/Cement	0.444
Heat Aggregate ☐ Cool Aggregate ☐ Hot Water ☐		Pour start time	7 a.m. on site

A	B	C	D	E	F	G	H	I
Aggregate Size	**1 m^3 dry weight (KG)** (SSD WEIGHTS FROM MIX DESGIN)	**% free water** (M.C. – Absorption Factor)	**Liters free water per m^3** C x B = D	**Batch Weight per m^3 of Concrete (KG)** (C x B) + B = E	**Batch Weight for (6) m^3 batch (KG)** E x (6) = F	**Liters free water (6) m^3** D x (6) = G	**(3) drop weights (KG)** F / 3 = H	**Accumulated drop weights (KG)** (if used)
Sand	703	3.6	25.3	728.3	4369.8	151.8	1456.6	1456.6
4.75-9.50	634	1.0	6.3	640.3	3841.8	37.8	1280.6	2737.2
9.50-19.0	422	0.5	2.1	424.1	2544.6	12.6	848.2	3585.4
Cement	390.5	kg/m^3		x (6) m^3 =	2343.0		781	
Air Ent.	148	ml/m^3		x (6) m^3 =	888.0			
WRDA	196	ml/100 kg		x (cem)/100 x (6) m^3 =	4592.3			
HRWR		ml/100 kg		x (cem)/100 x ( ) m^3 =				
		ml/100 kg		x (cem)/100 x ( ) m^3 =				

A	F	G	H	I	F	G	H	I
Aggregate Size	**Batch Weight for (7) m^3 batch (KG)** E x (7) = F	**Liters free water (7) m^3** D x (7) = G	**(3) drop weights (KG)** F / (3) = H	**Accumulated drop weights (KG)** (if used)	**Batch Weight for (8) m^3 batch (KG)** E x (8) = F	**Liters free water (8) m^3** D x (8) = G	**(3) drop weights (KG)** F / (3) = H	**Accumulated drop weights (KG)** (if used)
Sand	5098.1	177.1	1699.4	1699.4	5826.4	202.4	1942.1	1942.1
4.75-9.50	4482.1	44.1	1494.0	3193.4	5122.4	50.4	1707.5	3649.6
9.50-19.0	2968.7	14.7	989.6	4183.0	3392.8	16.8	1130.9	4780.5
Cement	2733.5		911.2	5094.2	3124.0		1041.3	5821.8
Air Ent.	1036				1184.0			

9.5 mm	Weight (kg)	Tare (kg)	Net Weight (kg)	% Retained	% Passing	% Stone Mix	%
Total Weight	7.00	0.50	6.50				
12.5 mm	0	0.50	0	0	100	60	60
9.50 mm	3.25	0.50	2.75	42.3	57.7	60	34.6
4.75 mm	6.36	0.50	5.86	90.2	9.8	60	5.9
2.36 mm	6.81	0.50	6.31	97.0	3.0	60	1.8
1.18 mm	6.84	0.50	6.34	97.6	2.4	60	1.4
0.300 mm	6.88	0.50	6.38	98.1	1.9	60	1.1

19.0 mm	Weight (kg)	Tare (kg)	Net Weight (kg)	% Retained	% Passing	% Stone Mix	%
Total Weight	12.00	0.50	11.50				
25.0 mm	0	0.50	0	0	100	40	40
19.0 mm	3.36	0.50	2.86	24.9	75.1	40	30.0
9.50 mm	8.59	0.50	8.09	70.3	29.7	40	11.9
4.75 mm	11.01	0.50	10.51	91.4	8.6	40	3.4
2.36 mm	11.86	0.50	11.36	98.8	1.2	40	0.5

37.5 mm	Weight	Tare	Net Weight	% Retained	% Passing	% Stone Mix	%
Total Weight							
50.0 mm							
37.5 mm							
19.0 mm							
9.50 mm							
4.75 mm							

	9.5	19.0	37.5	Total	Spec
50.0 mm					
37.5 mm					
25.0 mm	60	40		100	100
19.0 mm	60	30.0		90	90-100
9.50 mm	34.6	11.9		46.5	20-55
4.75 mm	5.9	3.4		9.3	0-10
2.36 mm	1.8	0.5		2.3	0-5

Moisture

	Weight (g)	Tare (g)	Net Weight (g)	
Wet Sand	161.5	20	141.5	A
Dry Sand	155.6	20	135.6	B
	Wet - Dry		5.9	C

$\frac{C}{B} = \frac{5.9}{135.6}$ = 4.35% (M.C.) - 0.746% (A.F.) = 3.6% (Free H_2O)

WRDA	5358				6123.0			

SAND	Weight (g)	Tare (g)	Net Weight (g)	% Retained	% Passing	Spec
Total Weight	155.6	20	135.6			
4.75 mm	21.4	20	1.4	1.0	99.0	95-100
2.36 mm	36.8	20	16.8	12.4	87.6	
1.18 mm	60.0	20	40.0	29.5	70.5	45-80
0.600 mm	91.6	20	71.6	52.8	47.2	
0.300 mm	124.4	20	104.4	77.0	23.0	10-30
0.150 mm	147.9	20	127.9	94.3	5.7	2-10
			FM→	2.67		
0.075 mm	152.6	20	132.6	97.8	2.2	0-3

Tested By *Robert. V. Young*

Figure 5-29: Concrete Gradation Test Report - Metric

(Rev 3/96)

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION

GRADATION TEST REPORT (SI)

Project Laconia — Field Test No. TCFA81276 & TCCA81276
Type of Material Fine & Coarse Aggregate — Reported 8/12 19 96
Reported by Ronald Tanner — Received by Lab ____ 19 ____
Report to: Project Files ☒ Lab ☒ Contractor ____ ☐
Sampled 8/12 19 96 At (town) F&S Transit Mix - Laconia
Source of Material J.J. Cronin – Laconia
Sample From Stock Bins — Pit ☐ Roadway ☐ Sta ____
Quantity (Represented or Estimate) 110 metric tons
Purpose / Location NE Backwall Bridge No. 080/130 — Item No. 520.01
Tested for: Gradation ☒ FM ☒ % Moisture ☒ on 8/12 19 96

Sieve (mm)	Coarse Aggregates and Gravels				
	Size 9.50 60 %	Size 19.0 40 %	Size %	Combined	Required
	% Passing	% Passing	% Passing	Results	Spec.
150.0					
90.0					
75.0					
63.0					
50.0					
37.5					
31.5					
25.0	60.0	40.0		100.0	100
19.0	60.0	30.0		90.0	90-100
12.5					
9.50	34.6	11.9		46.5	20-55
4.75	5.9	3.4		9.3	0-10
2.36	1.8	0.5		2.3	0-5
1.18					
0.300					
0.075 in Total					
% Fract'd Faces					
	Fine Aggregates and Sands				
	% Passing	Required Spec			
4.75	99.0	95-100			
2.36	87.6				
1.18	70.5	45-80			
0.600	47.2				
0.300	23.0	10-30			
0.150	5.7	2-10			
FM	2.67	2.65 +/- 0.2			
0.075 in Sand	2.2	0-3			
% Moisture	3.6				

Remarks: Moisture – 4.3% minus absorption factor of 0.746% = 3.6%
Meets requirements for: fine & coarse aggregates — See reverse ☐
Tested by: *Robert. V. Young* (Signature)

With all tests completed, the inspector can prepare the Concrete Batch & Delivery Record slips and instruct the batcher of the ingredient weights. Continuing the example begun above, using the Concrete Plant Work Sheet and the test results, the delivery slip would be filled out as in the example below.

This side to be filled out completely by the **Plant Inspector** for the first load of each truck size or when the mix changes due to moisture content variations, etc. However, only the top portion needs to be completed thereafter.

Time and amount of water are to be completed on every slip by the **Concrete Placing Inspector**. Other portions are to be completed at appropriate intervals.

(Revised 3/96)

NHDOT CONSTRUCTION BUREAU

CONCRETE BATCH & DELIVERY RECORD (SI)

Project Laconia Date 8/12/96
Truck # 33 Batch # 1 Size (A) 6 Sub-Tot 6 m^3
Mix Weights on Batch # 1 Total Delivered Water (B) 883.2 L
Total Delivered Weight (C) 13 780 kg Batch Time 6:13 AM
Robert V. Young 77188
Plant Inspector Employee Number

Complete the following for the first load of each truck size or when the mix changes
Source F&S Laconia Class AA
Location of pour Bridge #082/130, backwall
Scale inspected date 2/28/96 Water Temperature 11 °C
(Moisture = M.C. - Absorption Factor)

	Moisture		Free Water		Batch Weights	
Sand	3.6	%	151.8	L	4369.8%	kg
4.75–9.50	1.0	%	37.8	L	3841.8	kg
9.50 – 19.0	0.5	%	12.6	L	2544.6	kg
		%		L		kg
Water added at Plant	681	Liters x 1 kg/L =			681	kg

Total Delivered Water 883.2 L (B)
Cement Wt. 2343 + Pozzolan ____ = 2343 (G)

Total Delivered Weight (C) 13 780 kg

	Kind		at plant		
Air Entrainment	Kind	Darey	at plant	888	mL
WRDA	Kind	Hylol	at plant	4592	mL
HRWR	Kind		at plant		mL
	Kind		at plant		mL

Weather Fair Air Temperature 5 °C
Job Arrival 7:10 AM
Start Mixing 7:15 AM R.P.M. 11
Start Discharge 7:25 AM
Finish Discharge 7:35 AM
Water added at site 37.9 L x 1 kg/L = 37.9 kg (D)
Total Delivered (B) 883.2 L
Total Water (F) 921.1 L / size (A) 6 = 153.5 L/m^3

Concrete Temperature 13 °C Slump 50 mm Air 4.0 %
Cylinder Numbers 123, 124, 125
Unit Weight:
Weight 19.23
- Bucket 3.38 Conc Wt 15.85 = 83.4 kg/m^3 (E)
Conc Wt. 15.85 Volume 0.19

Yield: $\frac{(C + D)}{(E \times A)} = \frac{13\,780+37.9}{83.4 \times 6} = 27.6$

W/C Ratio: $\frac{(F)}{(G)}$ $\frac{921.1}{2343}$ = 0.393 microwave: ____

Ronald Tanner 56627
Concrete Placing Inspector Employee Number

Figure 5-30: Concrete Batch and Delivery Slip – Metric

P. Concrete Plant Batching:

Immediately prior to the batching of materials, the inspector shall verify that all trucks scheduled to deliver to the project are back spun and display a valid inspection sticker issued by the Bureau of Materials and Research. The inspector then continues to oversee the entire plant and stockpile operation in order to spot irregularities that may affect the consistent quality of the concrete product. Batching plants can be classified into three categories: manual, semi-automatic, and automatic. Only a few manual plants are still in existence and approved to be used for state work. Most plants that are being used are automatic or semi-automatic. The batcher controls all the functions necessary to complete the weighing and loading in the manual plant. This type of plant is subject to human error on every operation; therefore, the inspector should watch it carefully. A semi-automatic plant is governed by controls that are actuated in a certain sequence to complete the batching cycle. An automatic plant is considered one where the complete batching cycle is set in motion by a control button that may be located remotely from the plant.

The inspector must record the date on which the scales were tested on the delivery slip. The scales are typically checked and approved annually by authorized Department personnel and usually have a small seal on them with the date of last inspection. The plant inspector also must check the scale system. A scale mechanism basically is a series of compound levers arranged to balance a load of thousands of pounds (kilograms) at one end with a calibrated balancing weight of comparatively few pounds (kilograms) at the other end. The ratio of the lever system is on the order of 100 to 1 or more. The levers pivot on knife-edges that bear on flat blocks. The knife-edges and bearing blocks are made of very hard steel, which is also quite brittle. Good procedure would call for a daily visual check of all pivot points and constant observation of the dial or telltale indicator during batching operations. Erratic movement of these would suggest binding in the mechanism.

The possibility of cement shortages due to cement sticking in the weigh hopper has been well recognized by experienced construction personnel for some time. Another problem is not so well known. It involves the use of air pressure for the movement of cement to the weighing hoppers during delivery. It appears there are thousands of batching bins in use that are designed for air pressures of about 15 psi (100 kPa) using a 3”– 4” (75 mm to 100 mm) vent. Some cement trucks are now using pressures of up to 44 psi (300 kPa) and unless the vents are increased to 8” – 10” (200 mm to 250 mm) the increased air pressure may give a false reading on the scale. A quick check to see whether higher air pressures are giving false scale readings can be made by filling the cement hoppers and allowing them to stand for 10 minutes. If the scale indicator moves during this 10-minute interval it can be assumed that air pressure is affecting the scale reading and that additional venting is needed. The inspector should then request that cement delivery cease until the batching of cement into the load has been completed.

Water metering or measuring apparatus should be checked periodically to see that only the required amount is batched. Leaking valves can result in extra water entering the mixer. Weigh bins should be checked frequently to see that they are discharging completely and that material is not hanging up in the corners. The addition of the air-entraining agent should be checked to make sure that the intended amount actually gets into the batch and should be dispensed so that it goes into the sand and not on the stone.

Other admixtures, such as water reducers and retarders, should also be carefully watched. The dosages of these may be altered during the placement based upon on-site test results so the inspector should ensure that the batcher is notified of any changes.

Q. Concrete Plant Cement Sampling:

The final requirement of the plant inspector can be accomplished during one of the loading batches. A sample can of cement along with the cement test report or mill test report should be submitted to the lab (Materials & Research). This can be done by sending the sample with one of the transit mix drivers to the job site to be given to the on-site state inspector. If the plant inspector returns to the same plant the next day, or on a succeeding day, and no new shipment of cement has been received, the new sample may be tagged bearing the same information regarding shipment number, supplier, etc. This information would be copied from the plant's copy of the mill test report.

The plant inspector has an important function and must do everything possible to expedite changes and adjustments in the mix requested by the contract administrator during the pour. Any changes in the mix should be noted on the back of the delivery slips for future reference and documentation. And, finally, before leaving the batch plant, testing equipment should be cleaned and put away so that it will be ready to use by the next plant inspector and not left as an unsightly mess for the batcher.

The plant inspector should not leave the plant until he/she is sure **all** concrete that the project needs has been batched, regardless of the amount originally ordered.

R. Placing Concrete:

The contract administrator should encourage the contractor to have a worker meet and guide the concrete trucks. The pouring inspector should talk to the driver to inform him/her of the specification limits for slump. During the first round of trucks, until drivers become familiar with slump and consistency requirements, the placement inspector should work closely with the delivery personnel. Before placing initial concrete in the form, an inspector should complete the field tests for slump and air. All other tests should be taken in the middle of the load. The placement inspector should check the initial load for proper placement and vibration to establish proper procedures for the remainder of the pour. Old concrete at construction joints should be wetted along with the forms prior to commencing placement; however, water should not stand in the bottom of the forms. This prior wetting becomes very important in warm weather when backing up granite curb so that the granite does not draw water out of the concrete. For proper sampling procedures, see "Method of Making and Curing Concrete Cylinders" in Division 700 of this manual.

1. Underwater Concrete Placement:

   Placement of concrete under water must be accomplished so that cement will not be washed out of the mix by using a rigid steel tube called a tremie or by using a pump truck with rigid rather than flexible hose at the discharge end. At no time shall concrete be allowed to fall through water. The lower end of the tremie/pump must always be kept embedded in previously placed concrete. The tremie/pump can be moved laterally like a pendulum by first charging it with concrete, moving the top of the tremie/pump to the new location, and lifting and discharging the concrete. It is important to monitor the elevation of the bottom of the tube to ensure that it is not lifted out of the concrete during the discharge lift. The depth of the tube can be easily monitored by marking elevations on the tremie/pump. During the discharge lift, the bottom of the tube will swing to the new location in the concrete. It may be necessary to remove the tremie/pump from the concrete in order to move over a brace to another part of a cofferdam. Upon reentry, water can enter the tube. Therefore, a reseal must be made and care must be used not to flush water through previously placed concrete causing segregation. It is possible to close off the end of some tremie/pump hoses to water by duct taping a piece of polyethylene over the end of the tube. This plastic will keep water out. The pressure of the discharging concrete will break the tape after it has been placed in the previous layer of concrete. This allows the tremie/pump to continue concrete discharge without any flushing action. Basketballs are also commonly used to block the hose during reentry into the water. Concrete placed under water must not be vibrated or agitated. Cofferdams in running water must be tight enough to prevent loss of cement due to current action.

Competent supervision is of primary importance in any operation as complicated as an underwater concrete tremie placement. Concrete should be deposited under water only under the immediate supervision of project personnel. A vital part of the supervision lies in checking out preparations for a tremie operation. Elevation references should be set at several points around the cofferdam. A probe, level rod or sounding line should be on hand to check concrete elevations during placement.

2. Footing Concrete Placement:

   No footing concrete should be placed until the foundation has been inspected for stability and load bearing properties; and has been approved by the contract administrator. Likewise, no concrete should be placed in any portion of the structure until field personnel have had the opportunity and sufficient time to inspect the reinforcement, forms, falsework, mud sills, and tell-tales; and found these elements all to be satisfactory.

3. Columns or Thin Wall Placement:

   When placing concrete in columns or thin walls where a comparatively small volume is required, it is quite easy to overload the forms with a fluid head of unset concrete. For example, assume that the initial set time of Portland cement is between 3 and 4 hours at 70 °F. If the forms are calculated to be capable of holding a 10-foot depth of unset concrete, then it would be dangerous to allow a pour build-up greater than about 7 feet per hour at 70 °F. At 50 °F, this should be reduced to about 5 feet per hour.

4. Segregation Prevention:

   Proposed placement methods and equipment should be checked in advance of work in order to spot possible trouble areas. Mix segregation can be caused by poor placement practice. When concrete is placed from a chute, when permitted, the coarse aggregate will tend to separate out and form a rock heap almost devoid of mortar, while the mortar runs off separately. Specifications require the chutes to be of metal or metal-lined (but not aluminum). Concrete placed carelessly into wall forms can be segregated by causing particles of coarse aggregate to strike form ties or reinforcing steel. Concrete should be placed in a wall or column form by means of a metal or rubber elephant trunk to eliminate dropping the concrete more than 5 feet, by specification.

5. Curing:

 Concrete surfaces should be checked frequently during the curing period to see that they are always wet. When polyethylene plastic sheeting is used to cover slabs for curing, the sheeting should be lifted periodically in places to ascertain that the concrete surface actually is wet. Presence of moisture on the underside of the sheeting does not necessarily indicate that the concrete is wet. During periods of cooling weather, the concrete is likely to be several degrees warmer than the thin plastic sheeting. This will result in a migration of vapor moisture from the warmer concrete to the colder plastic. Care must be taken to see that the plastic sheeting is secured against being removed or torn by wind or carelessness. All edges and laps between sheets should be weighted sufficiently to guard against wind-caused air circulation under the sheeting.

A critical stage of the curing period occurs as concrete is passing from the plastic to the hardened state. During this period, which begins about an hour after placement, it is essential that concrete be protected from movement, drying, and sudden temperature changes. Concrete placed in slabs such as sidewalks or bridge decks during periods of windy or unusually warm dry weather may be vulnerable to cracking or surface checking. Slabs expose a large area per volume of concrete, which permits extremely rapid drying under the above conditions. Finishing must keep pace with placement, followed by the application of curing compound, to prevent excessive drying of concrete surfaces. Once finishing is completed, curing compound can be immediately applied to hold moisture in the concrete.

6. Concrete temperature:

Although often considered separately, concrete temperature is an important consideration in the curing process. It must be measured and recorded at various times and locations to establish and document that specified temperatures and rates of temperature change are maintained. The following temperature measurements are necessary:

- Mixing water.
- Concrete at the time of placement.
- Surface of previously placed concrete.
- Concrete temperature during curing period.

For measurement of concrete temperature during the curing period, location of temperature measurement points must be carefully worked out in advance. Widely varying temperatures are usually found in different parts of a concrete mass. Lowest temperatures will be found at corners and near protruding reinforcement steel. Highest temperatures in the mass also should be observed as it may be the critical control item at the end of the curing period when temperature drop must be held to the specified 1 °F (0.5oC) per hour or less.

Temperature measurements in the concrete mass are made easily with thermometers inserted into wells or voids formed in the concrete at the time of placement. A good thermometer well can be formed easily by inserting a greased bolt through the form prior to concrete placement.

S. Hot Weather Concreting:

Contrary to general belief, there is more danger of trouble from concreting in hot weather than in cold weather. Of all the ingredients in concrete, water is the one that causes the most trouble in hot weather concreting. It evaporates faster, causing a rapid change in the concrete's volume, which leads to cracking. Therefore, the mix should be as dry as possible.

Cooling the mixing water down to 45 °F (7°C) can clip several degrees off the final temperature of the concrete. Adding ice to the mix instead of water will reduce the final temperature far more drastically. Steps can be taken to keep stored water for concrete cool. Storage tanks, piping, and trucks should be insulated and their exteriors painted white. Also, aggregate stockpiles can be sprayed for cooling by evaporation. Cement at 150 °F (66°C) or higher should not be used because of the possibility of it forming into balls by rapid stiffening if it comes into contact with damp sand or small quantities of water. See Paragraph 520.3.8.1 of the Standard Specifications for temperature requirements for placing concrete.

The advantages gained by keeping materials cool will be rapidly lost if the concrete is over mixed or cast in hot forms. Over mixing can be averted by scheduling deliveries of truck-mixed concrete so that the trucks do not have to wait at the job site.

Delays in finishing concrete must also be avoided. When the surface is finished, steps should be taken immediately to cure the concrete. Taking the above precautions to lower the concrete temperature in hot weather is especially important for large pours. The term 'heat of hydration' is applied to the heat generated by the chemical reactions that occur in setting concrete between the water and cement. The heat causes the concrete first to expand and then to shrink as it cools. If there is a temperature gradient across the concrete or the concrete is otherwise restrained, cracking may occur. The two factors that are most likely to cause excessive generation of heat are very large pours and high cement contents.

For large footings, piers, etc., the idea is to pour each structure all at once. If you can't keep the concrete cool and moist, then keep it warm. It is very important to minimize differential temperatures. The outside surface temperature of the concrete should not differ from the ambient air temperature by more than 36°F. In other words, do not cure the structure's surface with cold water if internal temperatures are high. This is to prevent thermal shrinking, which causes surface cracking.

The following formula provides a relatively close approximation of what the maximum internal temperatures of a large structure might be within the first 3 days of curing, when the heat of hydration is at its peak.

$$T_{MAX} = T_C + 23°F \cdot P/170$$

T_{MAX} = Maximum temperature of hydrating cement (°F)

T_C = Temperature (°F) of concrete when placed

P = Number of pounds of cement per cubic yard

For example:

Mix: 3000 psi, ¾" aggregate, cement = 611 lbs/cy

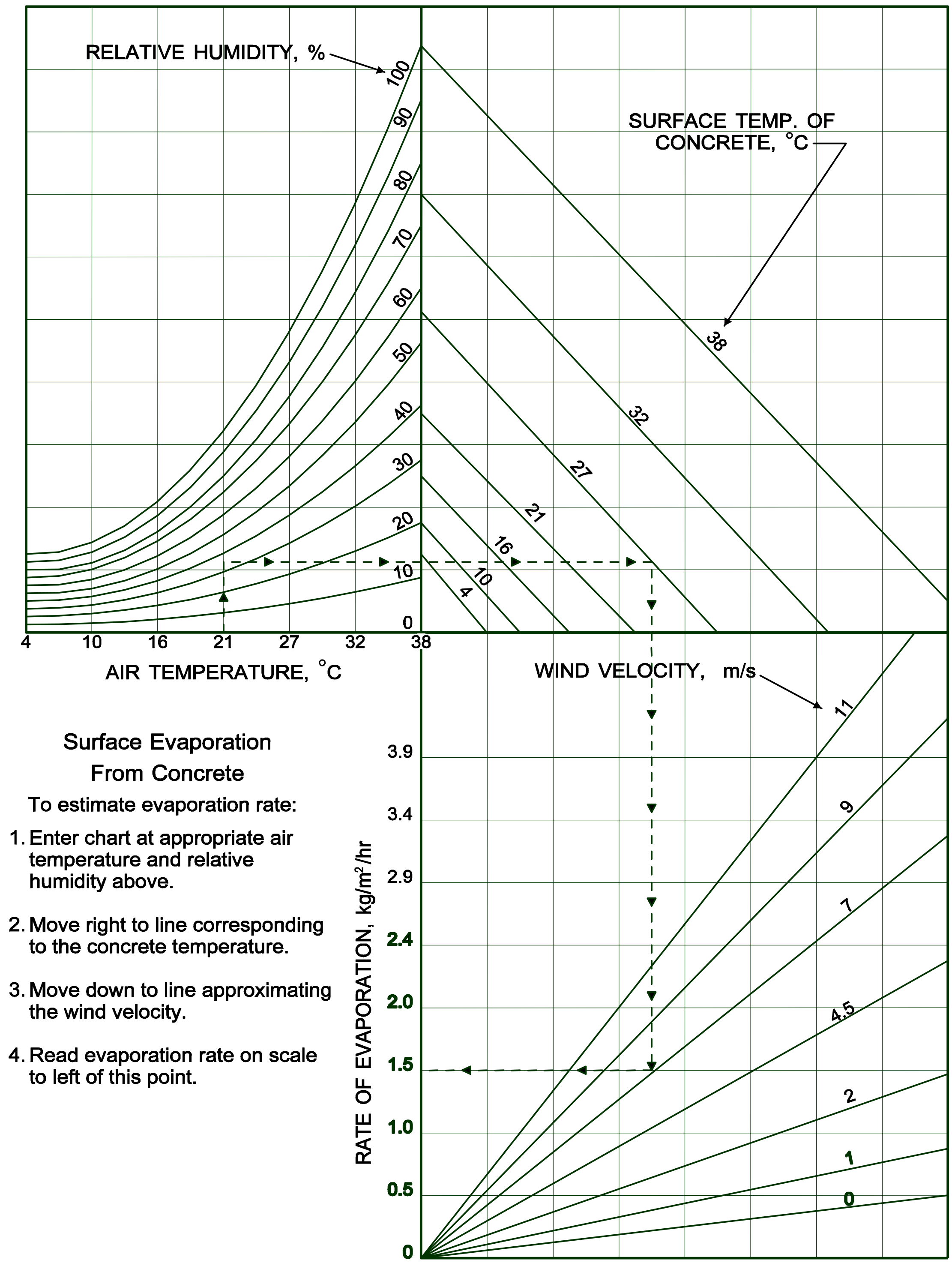
Concrete temperature measured at time of placement = 82 °F

$$T_{MAX} = 82\ °F + 23°F \cdot (611/170) = 165\ °F$$

Additives within the concrete mix may cause this temperature to vary.

Wind, accompanied by low humidity and high temperatures, can cause shrinkage cracks to appear in the surface of bridge decks and pavement. In this event, it may be necessary to erect shade or wind breaks, or both, in extreme conditions, or even to postpone placing bridge deck or paving concrete until more suitable temperature conditions prevail, or at night, in order to avoid too rapid drying of the surface and severe shrinkage cracking. A deck should have the wet curing materials placed as soon as possible behind the final floating.

The following metric and English unit nomographs are used to estimate the evaporation rate of surface moisture from concrete. They are primarily used to determine if extra protective measures will be needed during and after a concrete deck placement.



Recommended upper limit = 1.6 kg/m^2/hr

A sling psychrometer (relative humidity) and an anemometer (wind gauge) are available from the Materials and Research Bureau upon request.

Figure 5-31 (M): Nomograph for Calculating Evaporation from Free-Water Surfaces (Metric)

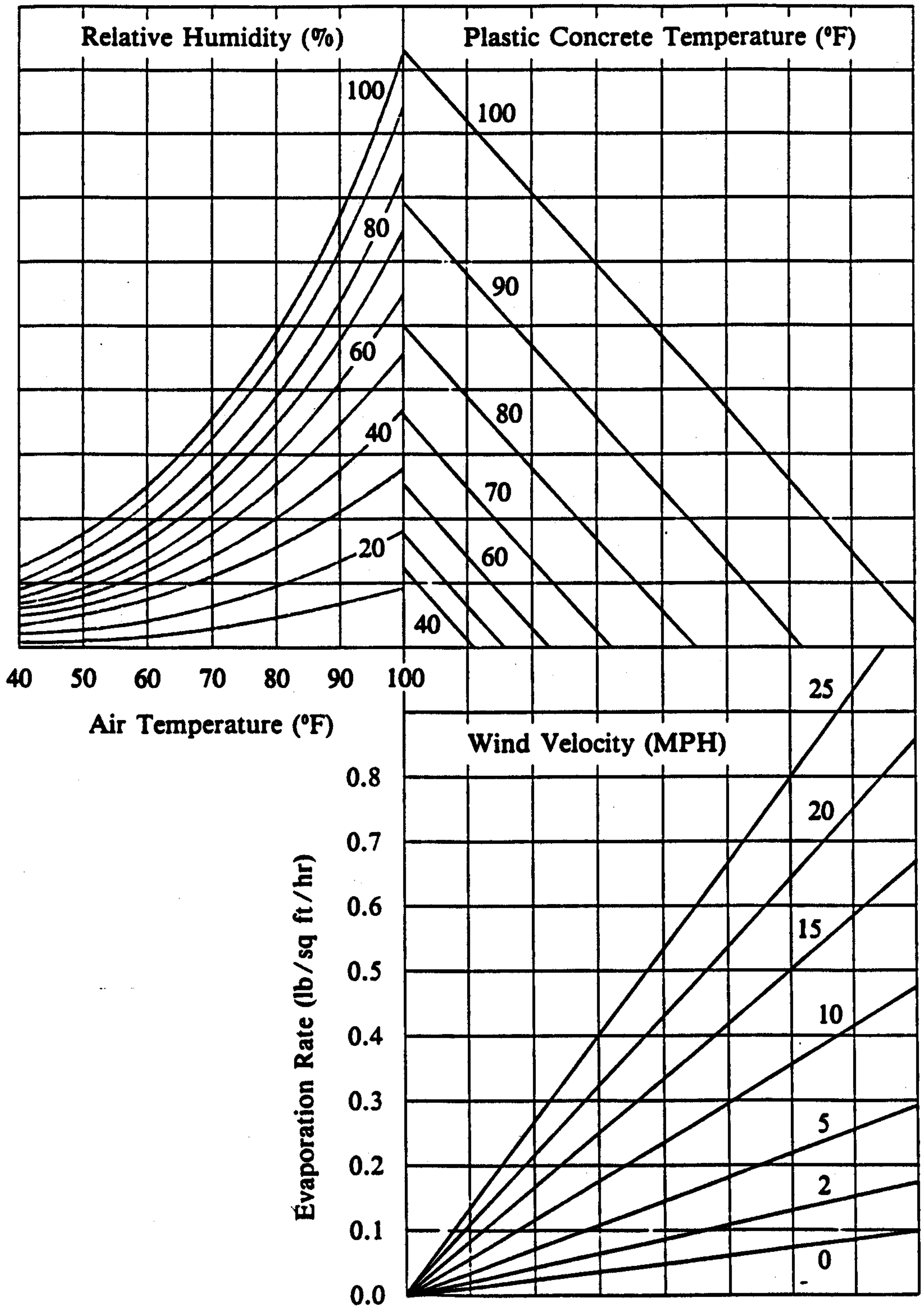


Figure 5-31 (E): Nomograph for Calculating Evaporation from Free-Water Surfaces (English)

T. Efflorescence:

Efflorescence is a deposit, usually white in color that sometimes appears on the surfaces of masonry. When it does, it spoils the appearance. Often it is apparent just after the structure is completed – the time when the builder, architect, and owner are most concerned with the appearance of the new structure.

A combination of circumstances is necessary for efflorescence to appear. First, there must be soluble salts in the masonry. Second, moisture must pass through the concrete to act as a vehicle that will pick up the soluble salts and carry them to the surfaces. If either of these two conditions is eliminated, efflorescence will not occur.

The efflorescence-producing salts found in masonry are usually sulfates of sodium, potassium, magnesium, calcium, and iron (ferrous), or carbonates of sodium, potassium, and calcium. Salts that are the chlorides of sodium, calcium, and potassium sometimes appear, but since these salts are highly soluble in water, the first rain will often wash them off the wall.

In most cases, salts that cause efflorescence come from within the concrete itself. However, sometimes chemicals in the masonry react with chemicals in the atmosphere to form the undesired efflorescence.

Another source of salts is the soil in contact with at-grade placements and retaining walls. If these sections are not protected with a good moisture barrier, the salts may be carried up to the first few courses above ground.

1. How to Prevent Efflorescence:

Since many factors can influence the formation of efflorescence, it is difficult to accurately predict the extent of efflorescence that may appear. However, as mentioned earlier, efflorescence will not occur if either (a) the soluble salts are eliminated from the wall or (b) water passage through the wall is prevented.

2. How to Eliminate the Salts:

Here are ways to eliminate efflorescence-producing soluble salts in masonry walls:

Use only cementitious materials for mortar that meet the requirements of AASHTO M 85.

Never use unwashed sand. Sand should meet the requirements of AASHTO T 71 and AASHTO M 6.

Never use masonry units that are known to effloresce while stockpiled. Use only masonry units of established reliability that pass the efflorescence tests in ASTM C 67.

Use clean mixing water free from harmful amounts of acids, alkalis, organic material, minerals, and salts. In some areas the drinking water might contain a sufficient quantity of dissolved minerals and salts to adversely affect the resulting mortar.

If walls of hollow masonry units are to be insulated by filling the cores, be sure the insulating material is free of harmful salts that may cause efflorescence.

Be absolutely certain that mixers, mortar boxes, and mortarboards are not contaminated or corroded. Never de-ice any masonry equipment with salt or antifreeze material.

Tools should be clean and free of rust, salts, and any harmful material. For example, do not use a shovel for any salt and reuse it for mortar sand without first thoroughly cleaning the shovel.

3. How to Eliminate the Moisture:

To eliminate the passage of moisture through masonry walls, these steps are recommended:

Correctly install flashings and copings to prohibit entry of water.

Install vapor barriers in exterior walls or apply barrier membranes or vapor-proof paint to interior surfaces.

Apply paint or other protective surface treatment to the outside surface of porous masonry units.

Tool all mortar joints with a V or concave shaped jointer. This compacts the mortar at the exposed surface and helps improve the tight bond of mortar to the edges of masonry units. Weeping, raked, and untooled struck joints are not recommended.

Carefully plan installations of lawn sprinklers or any other water source so masonry walls are not subjected to unnecessary wetting.

If architecturally feasible, use wide overhanging roofs to protect the walls from rainfall.

4. How to Remove Efflorenscence:

Most efflorescence can be removed by dry brushing. If this is not satisfactory, it may be necessary to wash the surface with a dilute solution of muriatic acid (5 to 10 %). Before an acid treatment is used on any masonry wall, the acid should be tested on a small, inconspicuous portion to be certain that there is no adverse effect.

Before applying acid to the surface, always dampen the wall with clear water. This will prevent the acid from being absorbed deeply into the wall where it may do substantial damage. After the acid treatment, the surface should be thoroughly flushed with clear water to remove all acid. Since an acid treatment may slightly change the appearance of masonry, the entire wall should be treated similarly to avoid discoloration or mottled effects.

A green stain may sometimes appear on buff or gray face brick or tile. This may be the result of vanadium or molybdenum compounds in the clay. Never treat such stains with acid, for the acid will react with these compounds to produce an insoluble brown stain that is extremely difficult to remove.

The proper method of removing the green stain is to wash the surface with a solution of 1 part sodium hydroxide crystals (lye) and 10 parts water. The wall should be first dampened by spraying with clear water and then washed with the sodium hydroxide solution. A thorough flushing with clear water should follow.

CAUTION: Rubber gloves and protective clothing should be worn by anyone using an acid or sodium hydroxide solution to clean masonry walls.

Another source of staining is from some granulated blast furnace slags now commonly used in concrete for improved permeability and strength properties. Project personnel might notice a bluish or greenish tinge immediately upon form removal. This coloration is temporary and will likely disappear completely over time as the concrete cures. No cleaning or treatment should be attempted.

U. Finishing Unformed Surfaces:

1. Tops of Footings, Walls, Piers, and Culvert Floor Slabs: These areas should be struck off and floated with a wood or magnesium float. The top surface of walls and piers should be struck off to a very accurate grade and floated to produce a relatively smooth, voidless surface. Culvert floor slabs should be struck off to the proper grade and floated with a wood float. There should be no low areas where water might collect. Those areas of floor slabs and footings which are to be covered with concrete in a successive placement such as piers or culvert stems should not be floated to a smooth surface but should not be left in an extremely rough state as it is difficult to get complete coverage of such rough surfaces when applying curing compound. For the rough surfaces at construction joints, there is too much surface area to cover with the normal rate of application of curing compound; consequently, these construction joints sometimes are not properly cured, resulting in weak concrete at the joint.

2. Bridge Decks: Failure to properly adjust screed rails or screed guides, improperly set joint headers or expansion joints, non-uniform consistency of the concrete, allowing the concrete to stiffen before finishing is accomplished, inexperienced workers, and poorly adjusted screeds and floats are some of the potential contributing causes of rough-riding bridge decks. The first straightedging should be done while the concrete is still in a condition in which corrections can be made. Any irregularities disclosed by the straightedging should be corrected immediately. Attention should be given to finishing the gutter lines on bridges, particularly on nearly flat grades, in order to preserve good longitudinal drainage.

V. Field Testing:

It is only by testing the concrete being used on the job that the most accurate determination can be made of the proper amount of each material to use in a mix. It is imperative that all project personnel engaged in inspection of concrete operations have a basic knowledge of the mechanics of mix proportioning. Factors to consider in evaluating concrete mixes can be summarized as rules of thumb.

Slump: Subtracting 1 gallon of water per cubic yard of concrete will generally decrease the slump by about 1 inch.

Air: Air entraining admixture may usually be increased or decreased without changing other contents of the mix in order to obtain allowable percent air. However, these changes in air can adversely affect workability and strength and should be controlled carefully. A change of aggregate or sand gradation might cause an air change. Therefore, changing the air entraining admixture dosage might help to correct the problem.

Workability: When a mix is unworkable and harsh, increasing the sand by 50 lbs/yd^3 (30 kglm^3) and decreasing the stone by 50 lbs/yd^3 (kglm^3) per cubic yard will result in a creamier consistency. If more sand is needed the adjustment can be repeated again, but usually in 50 lbs/yd^3 (30 kglm^3) increments. The workability of an over sanded mix can be adjusted the same way in reverse. The yield and strength should be checked after any rule-of-thumb change has been made.

Water/Cement Ratio: When adjusting concrete mixes as described in the above discussion, the water/cement ratio stated in the Specifications should not be exceeded.

It is often more feasible to make sure the water cement ratio is not exceeded by knowing the maximum amount of water per cubic yard for the given concrete mix ahead of time. Since the total delivered water is noted on the delivery slip from the plant, the placing inspector can quickly know how much water is allowed to be added at the project.

<u>Yield:</u> After the design mix has been prepared and approved, problems with yield can develop at the batching plant. Contamination of the aggregates by materials of different weights may result in changes in yield, since, in batching by weight, the volume of aggregate will change. For example, if lightweight organic material were to contaminate regular weight aggregates, a given weight of the aggregates would produce a greater volume than anticipated.

When the cement content is kept constant and yield increases, the cement factor is effectively lowered, and the result will be lowered strength. Conversely, if aggregates are contaminated with material of greater weight, yield will be reduced and strength increased. As an example, assume that a stockpile of structural lightweight aggregates accidentally becomes contaminated with regular weight aggregates. When the aggregates are batched, the weight will not supply nearly the volume anticipated and the yield will be reduced.

If yield is to be maintained, the moisture content of both the fine and course aggregates must be carefully watched. If weights in the mix design are predicated on aggregates in one condition of saturation (SSD) and they are batched in some other condition (without appropriate adjustments), the result will be a change in the aggregate volume batched and therefore in yield. Failure to adjust the amount of water in the mix will also cause loss of slump and workability or lowered strength.

This matter is especially critical in lightweight aggregate concrete where absorption can run as high as 12 percent. Unwatched, this can result in major variations in yield. For example, assume the following lightweight concrete mix design is based on Saturated Surface Dry (SSD) aggregate weights:

564	lbs	Cement
1300	lbs	Fine Aggregate (Sand)
900	lbs	Lightweight Coarse Aggregate
29	gals	Water
6	%	Entrained Air
2-4	inch	Slump

In the field, however, we find that the sand has a free moisture content of 5% and the lightweight coarse aggregate has a free moisture content of 10%. Applying a compensating adjustment, the mix should be batched with 564 pounds of cement, 1365 pounds (1300 x 1.05) of sand, 990 pounds (900 x 1.10) of coarse aggregate, and 10.4 gallons of water (155 pounds of free water equaling 18.6 gallons already exists in the material). If the mix is batched according to the weights given in the mix design (564 lbs, 1300 lbs, and 900 lbs, respectively), the actual cement and aggregate weights would be 564 pounds of cement, 1238 pounds (1300/1.05) of sand, and 818 pounds (900/1.10) of coarse aggregate. This would occur because there would be 62 pounds of water in the sand and 82 pounds of water in the coarse aggregate, for a total of 144 pounds (17.3 gallons) of extra water. Only about 11.07 gallons of additional mix water would be needed to produce the required 2-4 inch slump. In terms of yield, we would end up with a batch that yielded about 5% less than the design.

Air entrainment can also cause variations in yield. In regular weight concrete each percent of air entrained causes a 1 pcf (16 $kglm^3$) reduction in concrete weight. It also causes an increase in yield. When preparing a mix design for regular weight concrete, the effect of air entrainment on yield should be considered.

When trucks are forced to wait long periods before they can discharge concrete, there may be a reduction in yield due to loss of air and water. In this case, the Contractor must assume the responsibility for loss in yield, as well as the other degradations of quality that accompany long waits at the job site.

Other practices that can result in a loss of yield are over-vibration (this drives out air) and soupy mixes (increased water contents will result in lower unit weights). Any practice that affects the water or air content of a mix, such as adding water at the job or allowing concrete to dry out, will affect yield.

Certain types of concrete castings will also result in a loss of yield, even with excellent practices. For example, columns, high walls, mass concrete and other structural applications involving considerable concrete pressure will result in a loss of yield, often as much as 10 percent. This loss is due to the high degree of density achieved in such work and the attendant loss of air content. If the loss of intentionally entrained air is not excessive, this will naturally produce superior concrete. However, the loss of yield should be kept in mind and the order for concrete should take this into consideration.

To minimize problems with yield, the contract administrators should observe the following:

- Make sure that the mix design is consistent with a yield of ±27 cubic feet (cubic meter) of concrete per cubic yard (cubic meter) of designed mix. NOTE: The formula for this calculation is shown on the NHDOT concrete batch delivery slip (see Figures 5-27 and 5-30).
- See that the contractor orders sufficient concrete.
- See that the contractor schedules deliveries so that delays in discharging concrete from trucks will be minimized.
- Be sure forms are constructed mortar-tight.
- Be sure that the method of transporting and placing equipment will avoid over-vibration, segregation, and other problems.

Responsibility for variations in yield can be traced to conditions or practices before and/or after delivery of the concrete. In almost every case, the change in yield will be accompanied by changes in important characteristics such as concrete strength and durability.

Yield of the concrete as delivered at the job site can be checked by the inspector by regular measurement of unit weight. If this consistently varies more than 2 percent, an investigation should be made into such factors as air content, aggregate weight, or batch weights. When the unit weight varies considerably, it will often be found that the yield, as well as other concrete properties, also varies.

To check the actual volume of concrete being delivered to the job, divide the plastic unit weight of the concrete into the total number of pounds of all the mix ingredients.

Obviously, it is of great importance to maintain yield correctly. All parties benefit when yield is kept in bounds and all stand to lose when it is allowed to vary.

W. Design of Concrete Mix:

To adjust a mix design, it is best to be familiar with the initial design of concrete mixes. Data required to design a concrete mix would ordinarily come from field and laboratory testing. The following tables can be used in lieu of some items of field data.

Table 5-2: Volume of Coarse Aggregate per Unit Volume of Concrete - English

Maximum Size of Aggregate	b/b_o Volume of Dry Rodded Coarse Aggregate Per Unit Volume of Concrete for Different Fineness Moduli (FM) of Sand			
FM	2.40	2.60	2.80	3.00
3/8"1/2"3/4"1"1½"	0.50	0.48	0.46	0.44
	0.59	0.57	0.55	0.53
	0.66	0.64	0.62	0.60
	0.71	0.69	0.67	0.65
	0.75	0.73	0.71	0.69

Table 5-3: Water-Cement Limits in Different Classes of Concrete - English

Concrete	[1] Minimum Compressive Strength	Minimum Amount of Cement per Cubic Yard of Plastic Concrete		[2]Gallons of Water Air Entrained Concrete	
Class	PSI	Lbs	Bags	Gallons Per Bag	Water/Cement Ratio
[3]AA	4000	658	7.0	4.29	0.380
AA	4000	658	7.0	5.00	0.444
A	3000	611	6.5	5.23	0.464
B	3000	564	6.0	5.50	0.488
[4]T	3000	620	6.6	6.30	0.559

FOOTNOTES:

1. Minimum compressive strength expected at 28 days when cured in a moist room at a temperature between 65 and 75 °F.
2. If more water is required than shown, the cement factor shall be increased to maintain the desired water-cement ratio, or the concrete mix shall be redesigned.
3. When high range water-reducing admixture is used.
4. Fly-ash is not allowed in Class T Concrete.

Figure 5-32: Sample Concrete Mix Design - English

NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
BUREAU OF MATERIALS & RESEARCH
CONCRETE MIX DESIGN (ENGLISH)

Project: Laconia　　Lab No.: 391X96
Federal No.: NHS-018-2 (104)
State No.: 99999

Report to: ☒ Contract Administrator Ronald Tanner　☐ FHWA　☒ Lab
☒ Other ______　☐ Construction

Type of Mix: ☐ Non-Vibrated　☒ Vibrated　☐ Air Entrained　Slump 2-4 inch
Class AA ☒　A ☐　B ☐　C ☐　T ☐　P ☐　Max. Water/bag cement 5.0 Gal

Source of Fine Aggregate: Coastal-Farmington
Source of Coarse Aggregate: Coastal-Raymond
Type of Coarse Aggregate: Gravel ☐　Rock ☒　Chemical Admix. Req. WRA or Retarder
Size of Coarse Aggregate: #4 – ¾″
Fine Aggregate: FM 2.65　Absorption 0.746 %
Sp. Gr. (Sat. Surf. Dry) 2.632　Solid Wt. (A) 168.0 Lbs/CF
Coarse Aggregate: Rodded Weight (C) 106.5 Lbs/CF　Absorption 0.438 %
Sp. Gr. (Sat. Surf. Dry) 2.687　Solid Wt. (B) 180.8 Lbs/CF
Vol. Cse Aggr. per Vol. of Conc. = b/b_o= 0.64　x 27 CF/CY = 17.28 CF (D)
Wt. Cse Aggr. per CY of Conc. = (D) 17.28　x (C) 106.5 = 1840.3 Lbs (E)
Solid Vol. of Cse Aggr. 1840.3 (E)　/ (B) 180.8 = 10.18 CF
Solid Vol. of Cement = 7.0　bags x 0.478 = 3.35 CF
Solid Vol. of Water = 35　Gal / 7.49 = 4.67 CF
Volume of Air = 7.0　% x 0.01 x 27 = 1.89 CF
Total Solid Volume except Sand = 20.09 CF (F)
Volume of Sand = 27 - (F) = 6.91 CF (G)
Weight of Sand = (G) x (A) = 1160.9 Lbs
Ratio of Sand to Total Agg. ______ % by wt.　Yield Adj. to Design Mix ☐

BATCH WEIGHTS PER CY

Cement	658	
Coarse Aggregate (40%)	736	#4 – 3/8”
Coarse Aggregate (60%)	1104	3/8” – ¾”
Coarse Aggregate		¾” – 1½”
Fine Aggregate	1161	
Total Water	35	Gal/CY
Wet Density	146.3	Lbs/CF

Moisture content of Fine Aggregate and Coarse Aggregate should be determined and mix design adjusted prior to batching. Total weight of SSD Fine and Coarse Aggregates remains constant.

Respectfully James J. Marol　Concrete Supervisor

Respectfully Alan D. Angelos　Chief of Mat. Tech.

Date Reported February 21, 2002

For Department mix designs, changes in the fine aggregates should not vary from the design FM by more than ±0.2. If a greater variation occurs, a redesign of the concrete mix might be required. Following the procedure used on the original concrete mix design, a redesign can be computed up to and including the ratio of sand to the total aggregate using Tables 5-2 and 5-3.

Field checks should be made for the percent air entrained, weight of concrete, and yield after any redesign has been performed. Adjustments should be made if necessary. See Division 700 of this manual for concrete field sampling and testing procedures.

1. Factors Affecting Concrete Compressive Strength: The following rules of thumb apply:

For concrete of a given slump and a given maximum size of aggregate, the addition of one bag of cement per cubic yard, within the range of typically used cement factors, will increase the 28-day compressive strength by approximately 1000 psi.

For concrete having given proportions of dry ingredients, each increase of one inch in slump will be accompanied by a reduction in 28-day compressive strength of about 200 psi.

For properly proportioned mixtures of the same slump, doubling the maximum size of aggregate (say from ¾" to 1½") will increase 28-day compressive strength about 400 psi.

For properly proportioned mixtures, comparable as to cement factor and slump, air entrainment in usual amounts will affect strength approximately as follows:

- o In lean mixtures, no effect or slight increase in strength.
- o In mixtures of intermediate richness, a reduction in strength of 2 to 3 percent for each percent of air.
- o In rich mixtures, a reduction in strength of 4 to 5 percent for each percent of air.

X. Bridge Deck Construction Memory Jogger:

1. Schedule QC/QA Pre-placement Meeting:
   - Review Process Control Plan (PCP) – See Section 520.4 for more info.
2. Falsework and Deck Forms:
   - Adequacy of Design:
     - o Approved falsework plan on project and constructed falsework conforms to plan.
     - o Finishing equipment should be included with falsework plans.
   - Construction:
     - o Overall length and width of formwork for the span, checked and found to be correct.
     - o Deck form set to correct elevation with the blocking distances accommodating differences between actual and theoretical beam camber.
     - o All chamfer drip strips in place and securely nailed.
     - o Formwork free of construction debris.

    - Deck forms mortar-tight.
    - Bracing adequate at end dams and facias.
    - Utility structures not in interference with forms.
    - Deck drains, scuppers, and bridge rail anchor bolt assemblies in proper locations.
    - Safety check of equipment, guardrails, ladders, etc.

3. <u>Reinforcement Steel, Steel Joints, Expansion Material</u>:
    - Reinforcement Steel Bars:
        - Bars correctly spaced and securely tied.
        - Bars positioned to provide proper concrete cover and form clearance.
        - Bars clean and epoxy coating touched up.
    - End Dam Assemblies:
        - End dam assemblies checked for correctness in fabrication of dimensions, shapes, and straightness.
        - End dam assemblies set to proper grade.
        - Assemblies securely supported, securely attached, and shipping ties released.
    - Expansion material cork, Korolath, bond break paper, and filler joints correct in dimension and securely placed.

4. <u>Screed Guides:</u>
    - Adequacy of design:
        - Total load of all screeding equipment considered.
        - Rail supports designed to permit their removal above the top of the slab.
    - Construction:
        - Screed rails set to correct elevation and alignment.
        - "Dry Run" pass of screeding equipment over the screed rails performed, indicating space of rail supports to be adequate to prevent deflection of the rail at intermediate points.
        - "Dry Run" pass of screeding equipment over the screed rails indicates that the proper elevation tie-in has been made at span ends with the steel joints or the backwall of the abutment and deck thickness.
        - Dry Run is also performed to check proper concrete cover over the top mat of reinforcing steel.
        - Rails checked by eye for symmetry of cambered structure.

5. Equipment Planning:
    - Placement and Consolidation:
        - Concrete placement equipment, crane, chutes, pumps, etc. tested for operational condition.
        - Platforms in place to blanket the ground beneath the crane bucket during discharge of concrete from the truck or mixer.
        - Adequate number of properly functioning vibrators present with one or more in reserve.
        - Hand tools present.
        - Sample testing equipment available and adequate. Inclement weather station and functional curing box.
    - Finishing:
        - Screed checked for operational condition, particularly the straightness and cleanliness of the strike and screed bar edges.
        - Hand finishing tools and equipment present and free of hard paste.
        - Work bridge present, capable of facilitating access by finishers to any point on the deck.
        - Lighting equipment, including power source, present and checked when the finishing operations are likely to occur after dark.
    - Curing and Protection:
        - Equipment present for application of curing material.
        - Adequate covering present to protect concrete from damage by unexpected rainfall.
        - Adequate equipment present to maintain specified temperature ranges during cold weather concreting.

6. Personnel Planning:
    - Competent bridge superintendent to be present throughout the pour.
    - Adequate number of finishers scheduled.
    - Adequate number of vibrator operators.
    - Adequate number of laborers scheduled to place the concrete, operate the screed, and "serve" the finishers.
    - All personnel are well informed of their duties.
    - Adequate state personnel for on-site or plant testing.

7. Materials Planning:
    - Concrete:
        - Desired consistency of concrete specified to concrete producer.
        - Assurance from concrete producer that delivery will be adequate to maintain a continuous and uninterrupted placement.
        - System of communication exists between the job site and the batching plant.
        - The access road to the bridge site is in satisfactory condition to cause no delay.
    - Admixtures:
        - Manufacturer on Qualified Products List.
        - Adequate amount on hand to accommodate pour.

- Curing Materials:
    - Adequate quantity of concrete curing material present.
    - Temperature ports available.

8. Concrete Placement:
    - Preliminary handwork completed satisfactorily before machine progresses.
    - Finish machine and operator performance okay.
    - Vibration of mix properly performed.
    - Placement of concrete not too fast for machine.
    - Samples and test made, with any adjustments made accordingly.
    - Clearances of reinforced steel checked as finisher progresses.
    - Work bridge and finishing at sufficient interval behind finish machine.
    - Curing compound and wet curing materials applied at proper time.

Figure 5-34: Sample Concrete Pre-Placement Checklist

Project Laconia NHS-018-2 (104) Location So. Abut & Wing Footings

Computed Quantity 74 CY Date Placed August 12, 1995

Date	Initials	Description	Comments
8/11/95	MB	1. Vertical and horizontal alignment	
8/10/95	MB	2. Exposed face of forms (dented or damaged, oiled, all holes plugged)	
8/11/95	MB	3. Form dimensions (tightened mortar tight & chamfer strips installed if necessary)	
8/11/95	MB	4. Tie screws & bracing (all dimensions)	
		~~5. Oil contraction joint~~	Nos. 5, 6, 7, 10, and 13 not applicable for this pour.
		~~6. Waterstop location & proper installation~~	
		~~7. Weeper pipe location & installation~~	
8/11/95	MB	8. Re-steel & proper laps (in place) line and grade	
8/11/95	MB	9. Clearance of re-steel and ties	
		~~10. Bridge seat location, grades, and proper location of re-steel for anchor bolt~~	
8/11/95	MB	11. Grades for top of concrete	
8/11/95	MB	12. Clean bottom of form	
		~~13. Tremie location~~	
8/12/95	MB	14. Curing compound and protective covering	2 available
8/12/95	MB	15. Working vibrators (rubber coated)	
8/12/95	MB	16. Thermometer ports	
8/12/95	MB	17. Key ways prepared	
		18. ______________________	

520.4 – QUALITY CONTROL / QUALITY ASSURANCE (QC/QA)

Quality is defined by our customers, the traveling public. It can be defined as the degree of excellence of a product or service, the degree to which a product or service satisfies the needs of the customer or the degree to which a product or service conforms to a given requirement or expectation. The level of conformance to the customers' requirements is the measure of quality.

NHDOT contracts that include "QC/QA" specifications also include Quality Based Price Adjustment Clauses. These specifications allow financial disincentives for work with quality that is marginally less than desired and incentives for work that is above and beyond the Department's expectations. Here, the responsibility for providing the control of material quality is left in the hands of our contractors, subcontractors, suppliers, fabricators and manufacturers. Quality Control (or Process Control) is a comprehensive, systematic and continuous approach to producing and placing consistent materials that will perform within specification limits. Quality Assurance is a system for measuring the material quality provided based upon a statistical analysis of random testing of the finished product performed by the Department.

As many aspects of a product cannot actually be quantified so as to be measured and evaluated it is still necessary for NHDOT personnel to be actively engaged in overseeing the work. While it is true that the contractor is solely responsible for some quality control processes, it is essential for NHDOT personnel to actively participate in the planning and execution of the construction. Ensuring that quality is built into the project while it is happening is often the only means of obtaining the desired results.

Under QC/QA concrete specifications certain quality aspects (i.e., %air, water/cement ratio, concrete cover, strength and permeability) are identified to be controlled by the contractor and/or supplier in accordance with a quality control plan that they submit to the Department for approval. It is the NHDOT's responsibility to ensure that the plan contains all of the measures that will result in a final product meeting the criteria set forth in the specifications.

Contractors need to submit a project specific Process Control Plan (PCP) at least three weeks in advance for review by the Department. This plan outlines their organizational structure of personnel (i.e., QC Plan Administrator, production facility managers & staff, field technicians & inspectors, laboratory personnel, and consultant testing agencies); applicable specifications (i.e., standard & supplemental specifications, special provisions, AASHTO, ASTM, ACI, NETTCP, and project drawings); material control (i.e., sources, properties, mix designs, storage, and stockpiling); sampling and testing (i.e., lot & sublot sizes, sampling plan, sample identification, reporting, action limits); facility management (i.e., schedules, equipment, activities, control charts, non-conforming materials & corrective action, and production inspection); field management (i.e., schedules, equipment, control strip procedures, activities, material transport, delivery & placement, finishing & curing, corrective action of non-conforming materials, field inspection, and rejection criteria); and other plans (i.e., subcontractor, supplier).

The Process Control Plan (PCP) will be the document that all parties will refer to before, during and after the work is performed.

A successful Quality Control system does not rely just on the plan, but also continuous periodic formal inspection at all levels. It is imperative that those performing the work have a genuine knowledge and understanding of their responsibilities and how their actions affect the desired end product.

QC personnel are expected to test their products on a continuous basis to ensure quality requirements are being met. Department personnel, as stated earlier, will randomly sample and test the end product for Quality Assurance. The results of these QA tests will be used not just for pass/fail, but to make assertions about the Quality Level of the total material quantity. This Quality Level will then be used by Department personnel to make Pay Adjustments, positive or negative, to the bid prices.

The following analysis is used to determine Pay Factors for each characteristic being measured and is also outlined in the specifications:

1. Determine the average (X) of the test results.
2. Determine the standard deviation (s) of the test results.
3. Compute the Upper Quality Index:
   $Q_U = (USL - X)/s$, where USL is the Upper Specification Limit
4. Compute the Lower Quality Index:
   $Q_L = (X - LSL)/s$, where LSL is the Lower Specification Limit
5. Determine the Percent Within Upper Limit (P_U) from Table 106-1*.
6. Determine the Percent Within Lower Limit (P_L) from Table 106-1*.
7. Compute the Quality Level (total percent within specification limits):
   $QL = (P_U + P_L) - 100$
8. Go to Table 106-2* to determine the Pay Factor (PF) associated with this Quality Level

*NOTE: Table 106-1 and 106-2 may be found in the Supplemental or Standard Specifications.

The corresponding Pay Adjustment would be calculated as follows:

$$PA = PF \times Q \times UP$$

Where PA = Pay Adjustment
PF = Pay Factor
Q = Quantity in the lot
UP = Unit Bid Price

It should be noted here that when a Pay Factor computes to be less than 0.75, the Department will require that no pay adjustments be paid and that the contractor submit an engineering analysis showing either that the work will perform satisfactorily if left in place or a detailed plan for corrective action. The Department will review the analysis and, if the work is found to present little detriment to the serviceability of the structure, arrive at a negotiated settlement (credit) to be received from the contractor.

Figure 5-35: QC/QA Pre-Placement Meeting Agenda

PROJECT: ______________________________ DATE: __________

Location: ______________________________

Date of Placement ______________________________
Rain Date ______________________________
Start Time ______________________________
Total Quantity, # of Sublots ______________________________
Specification Date ______________________________
Delivery Rate, Route ______________________________
Method of Placement ______________________________
Expected Air Loss Pumping ______________________________
Direction ______________________________
Truck Access, Travel Time ______________________________
Washout Pit Location(s) ______________________________
Traffic Control ______________________________
Backup System ______________________________
Weather Station ______________________________
Lighting Placement ______________________________
Bad Weather Protection ______________________________
Curing System ______________________________
Sidewalks & Brush Curbs ______________________________
____
QC Personnel (Site, Plant) ______________________________
QA Personnel (Air, Cyls, µ) ______________________________
Air Meter Calibration ______________________________
Early Break Cylinders ______________________________
Testing Location, Access ______________________________
Info Air Test 1ST Load ______________________________
____
Split Samples (Schedule) ______________________________
Request Radar 3 Day Notice ______________________________
Add'l µ, Pyrex, Curing Box ______________________________
Distribution List & Fax Nos. ______________________________
____
Other Issues: ______________________________
____
____
____

SECTION 528 - PRESTRESSED CONCRETE MEMBERS

528.1 - GENERAL

The Materials and Research Bureau will serve as the central point of contact for precast/prestressed concrete issues that occur between the time of project award and the in-place installation of the members. Per an inter-department memo dated January 8, 2003 from bureau administrators to bureau personnel, the various parties' responsibilities are as follows:

A. Materials & Research Bureau:

1. Review and approve the concrete mix design. A copy of the approved mix design should be forwarded to the Bridge Design Project Engineer (PE) and District Construction Engineer (DCE).
2. Determine whether a test section is required and indicate if it is needed on the mix design approval.
3. Generate the Plant Inspection assignment letter to the Consultant Inspector. Copies should be provided to the Construction Bureau and the Precast Plant.
4. Forward one copy of the approved shop drawings to the Plant Inspector.
5. Notify the contract administrator if problems during production could affect the delivery schedule.
6. Request a repair procedure from the Precaster as necessary, and determine the level of inspection required for repairs. All structural repairs shall also be reviewed by the PE & DCE. Materials & Research will provide the approval to proceed with the repair and copy Bridge Design & Construction.
7. Conduct independent assurance inspection of Consultant Inspectors.

B. Construction Bureau:

1. Highlight the precast submission timeline requirements at the preconstruction meeting.
2. Six (6) copies of shop drawings shall be submitted to Construction (the detensioning sequence shall also be submitted on a separate sheet of paper – for informational purposes and not for approval). Forward all copies of shop drawings to Bridge Design for approval. Bridge Design returns 3 copies to Construction (1 for Contractor, 1 for contract administrator, 1 for Precaster), and 2 copies to Materials & Research (1 for Materials & Research and 1 copy for Inspector).
3. Forward the Contractor's mix design to Materials & Research. Coordinate Pre-Placement Meeting 45 days prior to casting.
4. Report all damage that occurs during erection directly to Materials & Research. Damage shall be thoroughly documented using Department forms, and pictures shall be taken of the damaged areas.
5. Include a copy of the Consultant Inspector Report in the project files.

C. Bridge Design Bureau:

1. Review and approve shop drawings; forward 5 approved stamped copies (3 to Construction and 2 to Materials & Research).
2. At the request of Materials & Research, Bridge Design will review damaged members to determine whether the structural integrity has been compromised and recommend a repair procedure if appropriate. Recommendations should be forwarded to Materials & Research with a copy to Construction.

3. Review Precaster repair procedures.

D. Consultant Plant Inspector:

1. Coordinate with the precast plant to determine casting schedule, and communicate this information to the Bureaus of Construction and Materials & Research.
2. Inspection: Members shall be inspected by the Plant Inspector during fabrication, at loading for shipment, upon arrival at the site, and during repairs required due to damage observed at any of these stages of inspection.
3. Reject unsatisfactory materials or workmanship during fabrication and prior to transport, in accordance with the specifications.
4. Report all damage during fabrication and transportation directly to M & R.
5. Notification process:
    - Members damaged during fabrication:
        - Report directly to Materials & Research.
        - Use the PCI NE Bridge Member Repair Guide as a reference.
        - Immediately notify Materials & Research of damage, which shall be thoroughly documented using Department supplied forms. Pictures shall be taken of the damaged areas. This information shall be provided to Bridge Design and Materials & Research as requested.
    - Members damaged during transport:
        - Plant Inspector reports to Materials & Research and informs contract administrator verbally.
        - Damage shall be thoroughly documented using Department supplied forms. Pictures shall be taken of the damaged areas.
        - Members shall be repaired only as approved, with all work performed by qualified personnel. Repairs shall take place in the presence of a Department inspector.
6. At the completion of the inspection assignment, the plant inspector shall complete and submit 2 copies of the final report that contains all records relevant to the assignment, such as daily logs, material reports, etc. The report copies shall be forwarded to the Bureau of Materials & Research, who will forward 1 copy to the Bureau of Construction.

E. Precasters:

1. Submit the shop drawings, concrete mix design, detensioning sequence and any necessary repair methods, as required by the specifications or as directed.
2. Notify Materials & Research, 14 days prior, of intent to cast so plant inspection can be arranged.
3. Notify Materials & Research 2 days prior to the actual start of casting.

SECTION 536 - EPOXY COATING

536.1 - GENERAL

This item consists of furnishing and placing epoxy resin protective coating systems on concrete surfaces. Some of these same materials are used as bonding agents for bonding new concrete to new or old concrete and for bonding new concrete to steel.

A. Typical applications in New Hampshire:

1. Waterproofing.
2. Concrete patching and repairing.
3. Resurfacing worn and damaged areas.
4. Bonding broken pieces of concrete back in position.
5. Bonding extruded curbs.
6. Bonding concrete placed in blockouts such as those used with modular expansion dams.

B. Other Applications for Epoxy:

1. Leveling bridge decks and sidewalks.
2. Leveling bridge seats for load distribution.
3. Bonding traffic buttons.
4. Bonding concrete to steel beams, dowels, and studs.
5. Making road surfaces skid resistant.

C. Reasons for Using Epoxy:

1. Solvent resistant.
2. Acid resistant.
3. Alkali resistant.
4. Salt resistant.
5. Waterproof.
6. Abrasion resistant.
7. Forms high-strength bonds with most materials.
8. Heat and cold resistant.
9. Cures rapidly (opening the area for traffic).

It should be noted that these materials are used both for protective coatings and as bonding agents. Generally when used as a protective coating, epoxy is a pay item; when used as a bonding agent, it is subsidiary to item 520. Check the plans and special provisions for any variations in this policy.

536.3 - CONSTRUCTION OPERATIONS

A strong reliable epoxy bond can only be achieved on a clean, dry concrete surface. As new concrete initially contains a considerable amount of water, some of which is surplus, the concrete must be permitted to age before any epoxy is applied. Epoxy should not be applied until most of the shrinkage has taken place. Epoxy mixes can be modified for use with fresh concrete, rapid curing concrete, and other concrete mixes, but the manufacturer's recommendations should be strictly adhered to in these cases.

A. New Concrete Preparation:

The surface of new concrete is always weak, no matter how strong the remainder of the concrete may be. This is a result of finishing the concrete. Vibration and troweling will bring the lighter components to the surface of the concrete. These lighter components, commonly called laitance, leave a smooth surface. Resist the temptation to leave it there. This laitance is perhaps ½ inch (1.3 mm) thick and is very weak in almost every respect. Unless this laitance is removed, it will limit the performance of anything applied to its surface.

Another surface condition often noticed, results from the use of compounds applied to wet concrete surfaces in the form of a membrane to retard water evaporation while concrete is curing. These curing compounds are often fatty oils or resinous materials of the type that will act as a parting agent for any subsequent adhesive or coating. Most curing compounds are more or less invisible unless a very careful examination is made. Even when traffic and weather have removed most of the curing compound, it is possible that some may be present on the more sheltered areas. Such curing compounds must be removed. Sand blasting may be required to remove these compounds and prepare the surface. Typically, curing compounds should not be used anytime a subsequent seal coat will be applied.

B. Old Concrete Preparation:

Old concrete has frequently deteriorated at the surface, leaving the surface weak. Even when the aggregate is exposed, its surface is often polished, which is an undesirable condition.

The deterioration of the concrete is often accelerated by the liberal use of de-icing salts during the winter. Removal of this weakened upper layer of concrete is desirable and essential prior to the application of epoxy.

Frequently, old concrete is stained by petroleum-based and other materials. These materials must be removed prior to the application of epoxy. Grease cutting detergents or, in the case of stubborn materials, solvents such as toluol or hi-flash naphtha may be used to remove the foreign materials.

C. Application of Epoxy:

Follow details in Section 536 of the Standard Specifications and the manufacturer's instructions when applying epoxy as a waterproofing sealer. If the contractor proposes to use a material other than what is exactly specified, check with the Construction Bureau office, Materials and Research Bureau, or the District Construction Engineer.

SECTION 538 - BARRIER MEMBRANE

538.1 - GENERAL

This item is of extreme importance to the life expectancy of a structure and should be very carefully inspected. Properly applied barrier membrane will prevent deterioration of concrete from salt and water intrusion.

A Department hired consultant inspector trained in barrier membrane inspection is usually utilized to assist and provide technical expertise to the CA for barrier membrane systems installed on bridge decks. Inspectors should be contacted early enough to allow for coverage during all aspects of membrane work, form beginning to end, starting with surface preparation of the bridge deck and ending with paving. Consultants should be contacted in the order of their contracts. This information may be obtained from the D.C.E. or Materials & Research Bureau. It is important that the authority of the inspector be defined early in the process. Inspectors are hired to work for the Department, specifically for the CA.

The barrier membrane specifications outline certain quality control tests to be conducted by the contractor and/or applicator during performance of the work. On newer projects, additional quality control procedures are documented in the membrane manufacturer's

Field Process Quality Control Plan. This plan must be approved by Materials & Research prior to the commencement of any membrane preparation or placement. Unless otherwise noted in the specifications or otherwise ordered by the Engineer, the Department's consultant inspectors should be conducting random quality assurance tests rather than more frequent quality control tests for the contractor.

All necessary membrane testing materials as noted in the specifications should be on site, such as, the moisture meter, Technical Guideline #03732 by ICRI, rubberized surface comparison chips and the adhesion tester.

538.2 - MATERIALS

Materials needed for standard sheet membrane consist of an adhesive primer, preformed waterproofing membrane sheet, and mastic. Liquid spray applied membrane consists of a primer, a two-part barrier membrane system (applied in two layers with usually a different color of membrane for each layer), and a final primer to bond with the asphalt. Torch applied membrane consists of a primer and an "asphaltic" sheeting membrane that should be placed by automation. Heat generated by automated or manual torches and the pavement is used to create the bond between the torch applied membrane and the primer and pavement, respectively. Hot rubber is often used to seal the curb lines for liquid and torch applied membrane. All components should be compatible as recommended by the manufacturer, and must be on the Qualified Products List. If the contractor wishes to use a product not on the Qualified Products List, a request and sample shall be submitted to the Materials and Research Bureau for approval. It is imperative that the correct type of membrane be used for specified, vertical and horizontal, surfaces.

538.3 - CONSTRUCTION OPERATIONS

Before performing any work, the contractor should be advised to protect all parts of the structure near the work area from being smeared with primer or mastic (i.e. granite curbing, catwalks & sidewalks, bridge railing, etc.). Primer and mastic smears, hardened liquid membrane, and charring from the torches from torch applied membrane are nearly impossible to clean satisfactorily and the contract administrator should insist that the waterproofing be applied in a workmanlike manner. If the contractor fails to comply with this request, additional measures must be taken such as covering exposed areas before proceeding with the work. The entire surface to be covered with barrier membrane should be cleaned of all foreign materials such as oil or grease. Any sharp protrusions shall be removed to create a reasonably smooth surface. Most new deck membranes require that the deck be blast cleaned to remove any laitance. The resulting surface should be checked against Technical Guideline 03732 by the International Concrete Repair Institute and concrete surface profile chips (rubberized replicas of typical surfaces), as required per specification 698.2.2.1. If the membrane is being applied on structures that were previously waterproofed, be sure to remove all existing waterproofing from the surface before applying new waterproofing.

No vehicles other than paving equipment should be allowed on the membraned surface prior to pavement overlay. Overlay equipment wheels and tires should be cleaned and free from stones and other material that could penetrate the membrane. A light dusting of Portland cement or talc may be used on the membrane in the wheel paths to prevent sticking during hot weather paving operations. If paving on top of the liquid applied membrane, a soapy water solution should be sprayed on the paver wheels to prevent delaminating the paving primer from the cured membrane.

The membrane is to be covered within 5 days as stated in paragraph 401.3.5.12 of the Standard Specifications. Since the application of a barrier membrane not only forms a water seal, but also an air seal, it may accumulate air bubbles. For standard sheet membrane, this trapped air can be released by taking a sharp pointed object and perforating the bubble at an angle near horizontal just prior to paving. The heat from the hot bituminous pavement will melt and reseal the puncture when rolled.

Any air bubbles in spray applied membrane must be released before applying the paving primer by cutting a slit and then sealing with a coating of the same liquid membrane mixture, usually applied with a print brush. Similarly, holes and holidays are fixed the same way.

SECTION 544 - REINFORCING STEEL

544.2 - MATERIALS

The manufacturer's delivery slips should be checked against the Department's bar list, and any discrepancies should be investigated and corrected, if necessary. Payment for this item will be according to the contract "F" item total if applicable. If the item is not a final pay (F) item, then it will be based on the corrected delivery slip totals.

544.3 - CONSTRUCTION OPERATIONS

The reinforcing steel should be stored on supports to prevent excessive sagging of the bundles and so that the steel does not come in contact with the ground. The contractor should be encouraged to store the steel with the shipping tags attached. When steel is stored for periods longer than two months, it should be protected from the weather with a covering. If plastic sheeting is the covering used, it should be draped over the bundles while still allowing air to circulate around the steel to minimize condensation under the plastic. Any covering placed over epoxy-coated resteel should be opague to protect the epoxy from exposure to the sun.

A. Resteel Markings:

When the bars arrive on site, check the identification marks stamped into the steel to be sure they correspond with specification and plan requirements.

The ASTM specifications for billet-steel, rail-steel, axle-steel and low-alloy reinforcing bars (A 615, A 616, A 617 and A 706, respectively) require identification marks to be rolled into the surface of one side of the bar to denote the producer's mill designation, bar size, type of steel, and minimum yield designation. Grade 60 bars show these marks in the following order:

1. Producing Mill (usually a letter)
2. Bar Size Number (#3 through #18)
3. Type of Steel:
   a. **S** for Billet meeting supplementary requirement (S 1) of (A 615)
   b. **N** for Billet (A 615)
   c. **R** for Rail (A 616) meeting bend test requirements of ASTM A 617, Grade 60 [per ACI 318-83]
   d.  for Rail (A 616)

e. **A** f o r Axle (A 617)

f. **W** for Low-Alloy (A 706)

4. Minimum Yield Designation: Minimum yield designation is used for Grade 60 bars only and may be either-one single longitudinal line (grade line) or the number 60 (grade mark). A grade line is smaller and is located between the two main ribs, which are on opposite sides of all bars made in the United States. A grade line must be continued through at least 5 deformation spaces, and it may be placed on the side of the bar opposite the bar marks. A grade mark is the 4th mark on the bar. Grade 40 and 50 bars are required to have only the first three identification marks (no minimum yield. Designation).

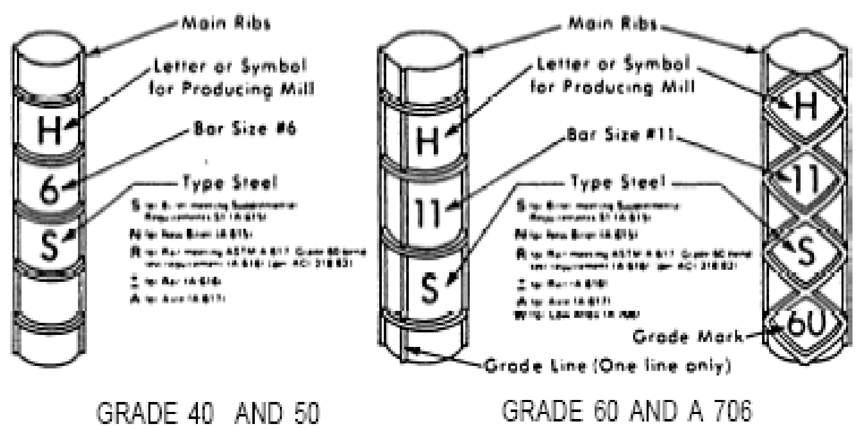


Figure 5-36: Markings on Reinforcing Steel Bars

Bar identification marks may also be oriented to read horizontally (at 90° to those illustrated above). Grade mark numbers may be placed within separate consecutive deformation spaces to read vertically or horizontally. For more information on marking requirements for reinforcing steel go online at: www.dot.ca.gov/hq/esc/construction/Manuals/BridgeDeck/appendix_3.pdf.

Some reinforcing bars are not marked to indicate manufacturer, size, and grade. If unmarked steel is received on the project, see that it is sampled and tested with satisfactory results prior to incorporating any unmarked bars into the work.

B. Resteel Placement and Support:

Reinforcing steel should be secured in its final position in the forms before the concrete placement begins. No reinforcement, such as dowels, should be permitted to be shoved into plastic concrete, since bond strength will be jeopardized.

Checking spacing between reinforcing bars in the forms can be a tedious undertaking, especially with closely spaced bars. Generally, it is acceptable to lay off a space of about three feet (one meter) and count the total number of bars in that space. For instance, if bars are called for at 9 inches (250 mm) on center for 4.5 feet (1.5 m), verify that there are six fairly regular spaces instead of measuring every 9 inches (250 mm). Minor variations in individual spacings are seldom of significance so long as the total amount of steel is correct.

Welding reinforcing bars to shear connectors, scuppers, end-dam assemblies, and rail bolt sets should not be allowed. The Bridge Design Engineers and District Construction Engineer should be consulted if any such requests are made by the contractor for legitimate reasons. Welding is not permitted on any main reinforcing steel. Welding, when authorized, should conform to Section 550 of the specifications.

Present specifications call for grade 60 (420) bars, but grade 40 (300) is sometimes specified at certain locations. Check your plans and special provisions carefully for usage of grade 40 (300) bars.

All chair and bar supports shall be estimated and furnished for bridge decks to allow for the minimum concrete cover of the reinforcing bars called for on the plans, and to ensure the spacing between supports does not exceed 4 feet (1.2 meters).

For transverse bars, two continuous lines of bar supports per bay shall be provided for spans (center line of beam to center line of beam) up to 9 feet (2.7 m), with lines positioned at the 1/4 and 3/4 points. For spans greater than 9 feet (2.7 m), use three continuous lines of bar supports per bay with lines positioned at the approximate 1/6, 1/2 and 5/6 points.

Bar supports are not intended to and shall not be used to support hoses for concrete pumps, runways for concrete buggies, or similar loads.

The nominal height of the bar support shall be taken as the distance from the bottom of the leg or runner wire to the bottom of the reinforcement. Variations of 1/8” (3 mm) from the stated nominal height shall be permitted.

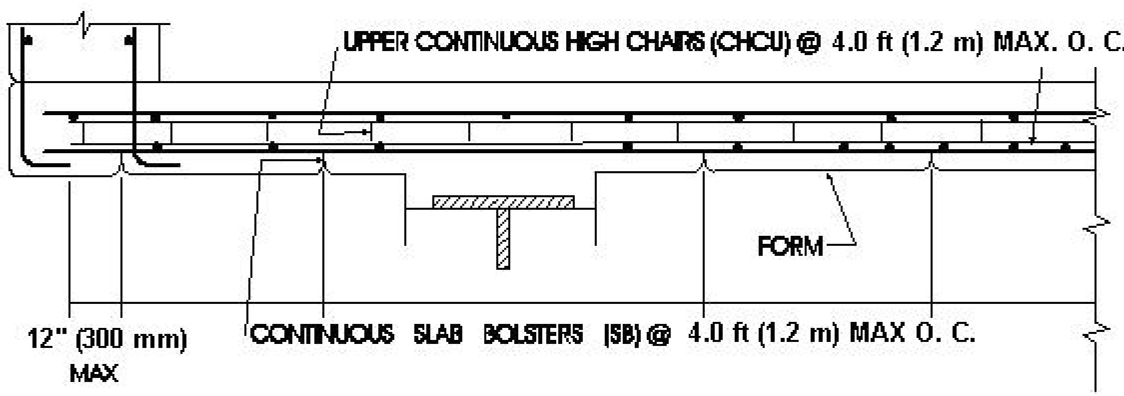
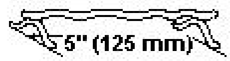


Figure 5-37: Slab Bolsters and Continuous High Chairs

The most widely used bar supports are standardized factory-made wire supports. Wire supports may be made from cold-drawn carbon steel or stainless steel wire. The lower portions may be provided with special rust protection by a plastic covering or by being made of stainless steel (bottom portion only).

The bottom reinforcement shall be supported on Type SB plastic protected or stainless steel protected supports. The end of the bottom supporting wire shall be overlapped to lock the last legs of the adjoining units.

All chair and bar supports used for the installation of epoxy-coated reinforcing bars shall meet standard specification requirement 544.2.5.4.

SYMBOL	BAR SUPPORT ILLUSTRATION	TYPE OF SUPPORT	STANDARD SIZES
SB		SLAB BOLSTER	3/4, 1, 1.5, and 2 inch heights in 5' and 10' lengths (20, 25,40,50 mm in 1.5-3 m lengths)
CHCU*		CONTINUOUS HIGH CHAIR UPPER	2 to 15 inch heights in 1/4" increments (50-280 mm in increments of 6 mm)

* TOP WIRE ON CONTINUOUS SUPPORTS, NOT OTHERWISE DESIGNATED AS CORRUGATED, MAY BE STRAIGHT OR CORRUGATED AT THE OPTION OF THE MANUFACTURER.

Figure 5-38: Steel Reinforcing Bar Supports

		Minimum Wire Sizes				
		Carbon Steel			Stain-less Steel	
Symbol	Nominal Height	Top	Legs	Runner	Legs	Geometry
SB	All	W5 Corru-gated	W2.9	--	W2	Legs spaced 5 in (125 mm) on center. Vertical corrugations spaced 1 in (25 mm) on center.
CHCU	2 in to 5 in (50to 125 mm) Over 5 in to 9 in.(125 mm to 225 mm)incl. Over 9 in to 15 in(230 mm to 380 mm) incl	W5.5 W5.5 W5.5	W5 W5.5 W8	W5 W5 W5	-- -- --	Legs at 20 degrees or less with vertical. All legs 8 in (200 mm) on center maximum, with legs within4 in (100 mm) of end of chair, and spread between legs not less than 50% of nominal height.

Reference: AASHTO M 32M

Figure 5-39: Minimum Wire Sizes of Bar Supports

Workers should be prohibited from walking on finished steel mats and encouraged to use walkways constructed on dunnage.

The correct clearance spacing of the reinforcing steel from the tension face of the concrete member is particularly important. Another important clearance check is the measurement of the deck reinforcing steel mat from the concrete surface. After setting the screed on the concrete finishing machine over several blocking points to final deck grade, run the machine over the entire mat and check the top mat clearance.

The epoxy coating on reinforcing steel is used to protect the steel from corrosion due to chemical reactions. This coating is to be abrasion resistant and flexible. The epoxy coating is applied by spraying the epoxy powder onto the bar and then heating the bar to cure the epoxy. Upon delivery, check the coating for any damage caused during transportation. Since the epoxy coating is flammable, bars should not be cut by burning. After cutting, the bars should be touched up with a repair kit supplied by the manufacturer. Use the same repair kit to touch up any subsequent damage to the coating. Repair kits generally require mixing up of a two-part epoxy then brushing the epoxy onto the exposed steel (the damaged area should be cleaned of all rust and contaminants prior to the application of the epoxy). Contractors may favor a spray applied epoxy. This "spray paint" type epoxy is generally not approved by the manufacturer, and as such, should not be used as a substitute to the manufacturer supplied repair kit. When reinforcing steel is epoxy-coated, plastic-coated tie wire and plastic-coated bolsters are required. The epoxy coating is slippery. Therefore, bars should be tied tight to prevent their movement. When compacting the concrete use only rubber or non-metallic vibrators to consolidate the mix. A metallic vibrator can cause damage to the epoxy-coating bars within the concrete.

For sampling of reinforcing steel see Division 700 of this manual.

SECTION 547 - SHEAR CONNECTORS

547.1 - GENERAL

The Standard Specifications contain sufficient information to inspect and test the installation of shear connectors. Spirals and structural shapes are usually shop installed, and since they are not used very frequently in New Hampshire, they will not be discussed.

547.2 - MATERIALS

At an early date, the contract administrator should call the contractor's attention to the requirements that must be fulfilled before ordering the studs as specified in paragraph 547.2 of the Standard Specifications. Before placement, the contract administrator should inspect the studs, making sure that they are the proper size, length, and diameter. If the steel camber is near the tolerance limit, check with the Bridge Design Engineer or the District Construction Engineer for a possible change in stud length.

547.3 - CONSTRUCTION OPERATIONS

A. Layout:

The contract administrator should check the contractor's layout prior to the field installation of shear connectors.

B. Workmanship:

The studs should be welded on the designated layout points. A maximum variation of 1 inch (25 mm) will be accepted, provided the adjacent studs are not closer than 2.5 inches (65 mm) center to center.

C. Work Area:

All reinforcing steel, mesh, and other materials and equipment that will interfere with the stud installation shall be removed.

D. Safety:

The specifications now allow studs to be shop installed. Note that when they are shop welded, the contractor must provide suitable staging for the contract administrator to safely take necessary elevations.

SECTION 550 - STRUCTURAL STEEL

550.1 - GENERAL

This section covers the broad field of furnishing and constructing the structural steel portion of structures. Steel bridges constitute the most important part of this section; therefore, the following paragraphs apply primarily to steel bridges but also apply, in part, to most metal structures (such as handrails, etc.). The contract administrator will have to determine what portions of the following paragraphs apply to some of the minor structures or structural components.

550.2 - MATERIALS

The contractor shall provide shop drawings of the steel through the District Construction Engineer to the Bridge Design Bureau for approval. The steel is then fabricated from the approved drawings under the inspection of a Department approved testing agency or inspector.

The contract administrator should inspect the steel for damage upon its arrival on the project and report to the Bridge Design Engineer in the Bureau of Bridge Design any damage during transport of structural steel before erection.

Before erection of the steel and any partial payment, the contract administrator should have received the following items:

1. Approved shop drawings.
2. Certificates of Compliance.
3. Mill Test Reports.
4. Inspection reports.
5. Erection Plan by NH Professional Engineer, received for documentation. This should be reviewed by the contract administrator and District Construction Engineer for constructability.
6. Subcontractor approval if applicable.

550.3 - CONSTRUCTION OPERATIONS

A. Bridge Seats:

1. Bearing pedestals are sometimes poured approximately ¼" (6 mm) lower than the plan elevation. After the bearing pedestals have cured, they should be ground level. Elevations are then taken at each pedestal and the difference between the actual and the plan elevations should be computed. Shims equal to the computed differences are then ordered by the contractor for use under the bridge shoes.

Sometimes bearing pedestals are finished to grade while placing and checked with a transit or level during the finishing process. Other times, the pedestals are finished slightly high (1/8" to ¼"), then ground down level to the design elevation. If this approach is used, it is important that the whole seat be ground down to avoid ponding on the bridge seat. See Figure 5-40 for a diagram of a ground level bridge seat.

2. After the bearing pedestals are confirmed level and the elevations are checked, a transit and tape or an electronic coordinate system should be used to accurately locate the centerlines of bearing and girders. NHDOT survey is only obligated to perform an initial layout of the bridge seats. Since this is usually done before the seats are poured,

the contractor is responsible for resetting any and all future points. Regardless of this fact, it is still very important for the CA and/or inspectors to double check the layout before any bridge shoes or steel are placed.

3. The contractor should mark anchor bolt holes for drilling using a bridge shoe template (typically made out of plywood). If anchor bolt holes are to be drilled and left in the winter, they should be filled with antifreeze and covered.
4. If the application of epoxy coating, water repellent, or concrete sealer is specified on the horizontal bridge seat area, the paint coatings should be applied before the bridge shoes are positioned.

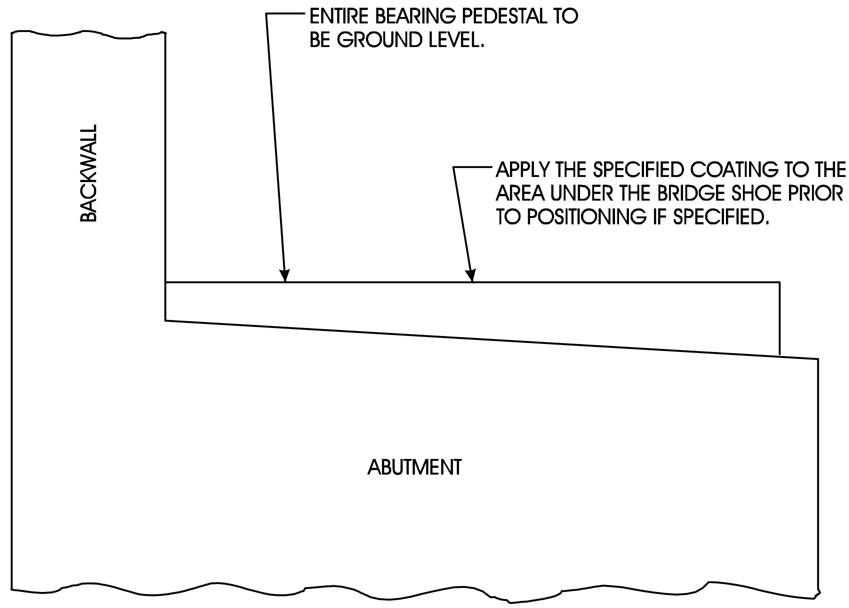


Figure 5-40: Leveling Bridge Seats by Grinding

B. Placing the Bridge Shoes:

1. When the contractor places bridge shoes on the bridge seats, check to see if the correct shoe is being used. Different abutments and piers may have different types of shoes (fixed or expansion), and different girders may have different sized or tapered shoes, together with various sized prefabricated bearing pads for each.
2. In most cases, do not grout shoe anchor bolts until structural steel is completely assembled, bolted up, and accepted. After erection is complete, and just prior to grouting the anchor bolts, the shoes should be positioned for the expected travel due to temperature changes and aligned with the axis of the stringer.
3. If the expansion bearings are of the rocker type, the rockers are adjusted according to the prevailing temperature so as to be vertical at the standard temperature shown on the plans. In the absence of a plan chart, use 1/8” (3 mm) for each 10^0 F (6 ^{o}C) of temperature per 100 LF (30m) of bridge. Rockers should be rechecked and reset, if necessary, at various intervals until they all stay in the same relative position. Additional checks will be necessary until the desired results are accomplished. Do not grout in anchor bolts until all rocker adjustments are complete.

C. Storage and Handling of Members:

Steel members should be stored off the ground on skids or cribbing. Beams and girders must be placed upright and stored securely so that they will remain upright. Beams and girders are rigid only in the plane of the web and have little resistance to lateral deflection or twisting.

1. While the steel is on the ground the contract administrator should inspect it to see that no paint or protective coating (unless specified) is on the surfaces that are to be in contact with concrete or at steel splice areas. If it is necessary to remove paint in these areas, they should be sand blasted.
2. Under no circumstance, should pin holes or bolt holes be used as hook-on points for lifting structural steel. Bolt holes are precisely machined to close tolerances. The material adjacent to a bolt hole can be mutilated when a single hole is used as the hook-on point to lift a heavy member. Beam clamps or straps are typically used for hoisting and positioning structural steel. Tag lines are generally affixed to the ends to give the contractor better control over swaying.

D. Steel Erection:

Shop drawings contain fabrication code markings, and will provide the contract administrator with erection information to identify beams, splice plates, diaphragms, scuppers, end-dam components, and the direction of north or east. The contract administrator should be satisfied that the erection procedure will be successful before beginning erection. Any questions should be referred to the Bridge Design Engineer and/or District Construction Engineer. Factors to be considered are crane capacity; footing for cranes and out-riggers, aspects of cranes working on barges; temporary bracing that may be required to support girders after they are erected; and possible hazards to traffic. During erection, the contract administrator should inspect the operation to ensure equipment and procedures adhere to the approved-plan. As each girder is placed it should be bolted up and sufficiently braced to keep from collapsing. With the increasing use of curved steel beams, the contract administrator should be aware that special erection procedures are necessary. A curved beam, unlike a straight one, will not stand unsupported on the bridge shoes. Therefore, the contractor must modify the erection procedure or use additional falsework.

E. High Strength Bolts:

High-strength bolts and nuts can be identified by markings described in Section 550 of the Standard Specifications. Bolts, nuts and washers should arrive on the job with a protective coating of oil. It is extremely critical that these bolts are maintained in their factory condition and sealed in the original shipping containers until the day they are to be used. The oily coating applied at the factory will help keep the threads from rusting, which could adversely affect the performance of the fastener during torquing. If the bolts are found to have any degree of rust on the threads, the contractor should be directed to relubricate bolt and nut threads with a wax or oil as approved by the manufacturer and the Bureau of Materials and Research. Due to the fact that final tensioning of all bolts should be done only after all structural steel is erected, it is recommended that the contractor field lubricate all threads prior to initial installation of a multi-span bridge. This recommendation is based on the assumption that the bolts may sit in structural members for several days if not for a couple of weeks. During this time the bolts become weathered and lose their factory applied lubrication. Without factory or field lubrication, the bolts cannot be tensioned properly.

When metric bolts are specified, ensure that only metric bolts are used. Because the size of shop-drilled bolt holes are based on the size of the bolt specified, it is very important that only the size and type of bolt specified is actually used. For example, if an M20 bolt is specified, do not substitute a 3/4 inch (19.05 mm) bolt even though the 3/4 inch bolt is only 1 mm smaller than an M20 bolt. The holes in the plates were sized for a 20 mm diameter bolt, not a 19 mm diameter bolt.

Proper tightening of high-strength bolts: The Standard Specifications require the use of direct tension indicator washers (DTIs). These washers have bumps machined into them that will compress once the proper bolt tension has been obtained. It is necessary for the contract administrator or his/her assistant to check the area under the DTI with a feeler gage to ensure that the DTI has been properly compressed without over compressing. The specifications should be checked to determine the required amount of DTI compression.

When using DTI's, care should be taken when field verifying or testing each rotational-capacity lot of bolted connections (employing the Skidmore). The inspector should take the time to assess the behavior of both the DTI and the bolt assembly when the connection is brought to both the verification tension and to the "no entries but visible gap" tension. On many occasions, the inspector might find it difficult to relate the recommended tension to the appropriate number of entries allowed. Also, it is very difficult to observe a "visible gap" once all entries of the DTI are closed. This is especially true when the DTI is coated or galvanized because the coating may flake making the "visible gap" condition undeterminable, thereby making the tension load in the bolt indeterminate.

If problems occur when relating required tension to the number of entries around the DTI, the inspector should.

Verify the Rockwell hardness (Rc) of the DTI (Rc< 35). The inspector should also verify that all bolts, nuts and washers in the rotational-capacity lot conform to specifications.

Check bolt assemblies for adequate lubrication.

Check bolts and nuts for dirt, burrs, defects or other material to include areas of built-up galvanizing that would prevent the inspector from getting satisfactory results during verification testing and inspection.

During inspection, if the inspector becomes satisfied with a particular single connection, he/she may physically measure the bolt reveal or "stick-out" of that connection and use this information to assist in determining the tension in other connections.

As a last resort, the inspector should reject the DTI lot if all requirements set forth in the specifications were performed during verification testing and results are not satisfactory.

Two other methods of tightening listed in the specifications are: (1) calibrated wrench tightening and (2) turn-of-nut tightening. As these methods are considerably less reliable, they should not be used in lieu of

DTI's in cases where DTI's are called for on the plans or in the contract specifications. Calibrated wrench tightening uses a torque-control wrench that cuts off when a pre-set torque is reached. This method is extremely unreliable if the bolts become rusted or are slightly out of alignment causing them to rub against the steel splice plates or diaphragms. This can result in a high torque reading but low tension. Turn-of-nut tightening can be accomplished with either a hand wrench or standard impact wrench. The usual source of power is compressed air. There must be an adequate pressure at the tool – an absolute minimum of 100psi (690kPa) for bolts 7/8" (22 mm) in diameter and smaller. For larger bolts, the pressure must be higher.

2. Calibration: No matter what method of bolt tightening is used, the use of a calibrating device to verify DTI and bolt performance is essential. The Standard Specifications detail the necessary testing. The calibrator, furnished by the contractor, and similar to that shown in the figure below, must be inspected by the Materials and Research Bureau in Concord prior to its use on the project.

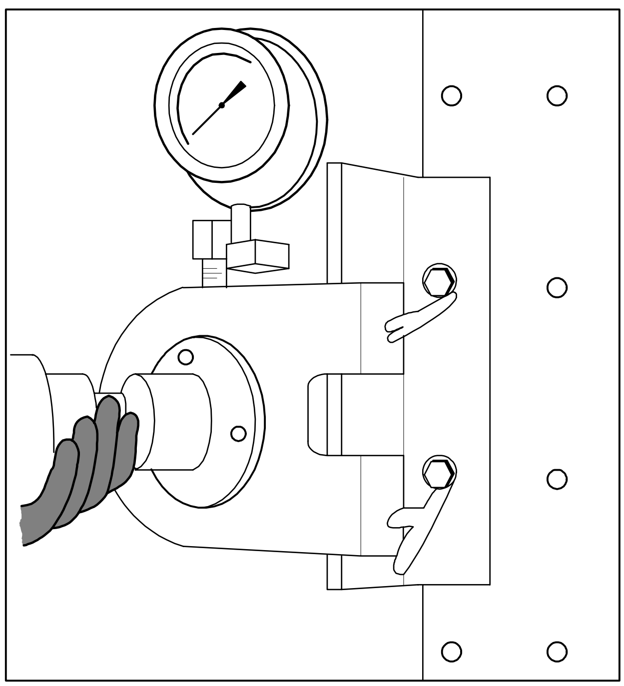


Figure 5-41: Bolt-Tension Calibrator (Skidmore)

The bolt-tension calibrator, also know as the "Skidmore," is a hydraulic loaded cell that measures bolt tension created by tightening. As the bolt or nut is turned, the internal bolt tension or clamping force is transmitted through the hydraulic fluid to a pressure gauge which indicates bolt tension directly. Older bolt-tension calibrators indicate tension in pounds; newer calibrators are dual dimensioned, giving the bolt tension in both pounds and newtons. The dial of the gauge may be marked to show the required minimum tension for each bolt diameter.

When torque-control wrenches are used, the contract administrator must insist that they be calibrated at least once each day by tightening not less than three bolts of each diameter from the bolts to be installed. The test bolts are not to be used for installation and must be discarded. The average torque determined by this calibration procedure may then be used to pre-set the cut-off device built into the torque-control wrench. The torque-control device must be set to provide a bolt tension 5 to 10 percent in excess of the minimum bolt tension.

The bolt-tension calibrator is also necessary to calibrate the hand-indicator torque wrenches, also to be furnished by the contractor, and is used by inspectors for checking torque as it relates to tension after tightening by either the calibrated wrench method or the turn-of-the-nut method. Again, if either of these methods is used, a close examination of the condition of the bolts must be performed regularly. Any rust or imperfections can significantly impact the torque-tension relationship.

3. Installation Sequence: Regardless of the method used to tighten high-strength structural bolts, the sequence of operations is basically the same. First, holes in splice plates and beams are "faired up" with enough drift pins to maintain dimensions and plumbness. Next, sufficient high-strength bolts of the proper size are installed to hold the connection in place. Only hand tightening is required at this point. Since these bolts will remain in place as permanent fasteners, washers, if required, should be installed with the bolts during fitting-up. The balance of the holes are now filled with bolts and assembled with nuts and washers. Fascia beam splice connection bolts should be inserted with the bolt heads exposed to view (nuts in or up). The test bolts may not be used for installation and must be discarded.

At this time, elevations should be taken on top of the beams at splice connections to determine whether adjustments of beam elevations are needed. Correction of these elevations will keep blocking distance variations to a minimum.

Some contractors may contend that once some bolts and drift pins are placed the splice elevation cannot be adjusted. This is simply not true. Splices can be lifted and then tightened in place or they can be lowered by loosening and lifting the adjacent splice. Desired elevations can be substantially obtained by the above procedure. Figure 5-42 depicts a splice and the calculations for adjusting the beam elevation at that splice.

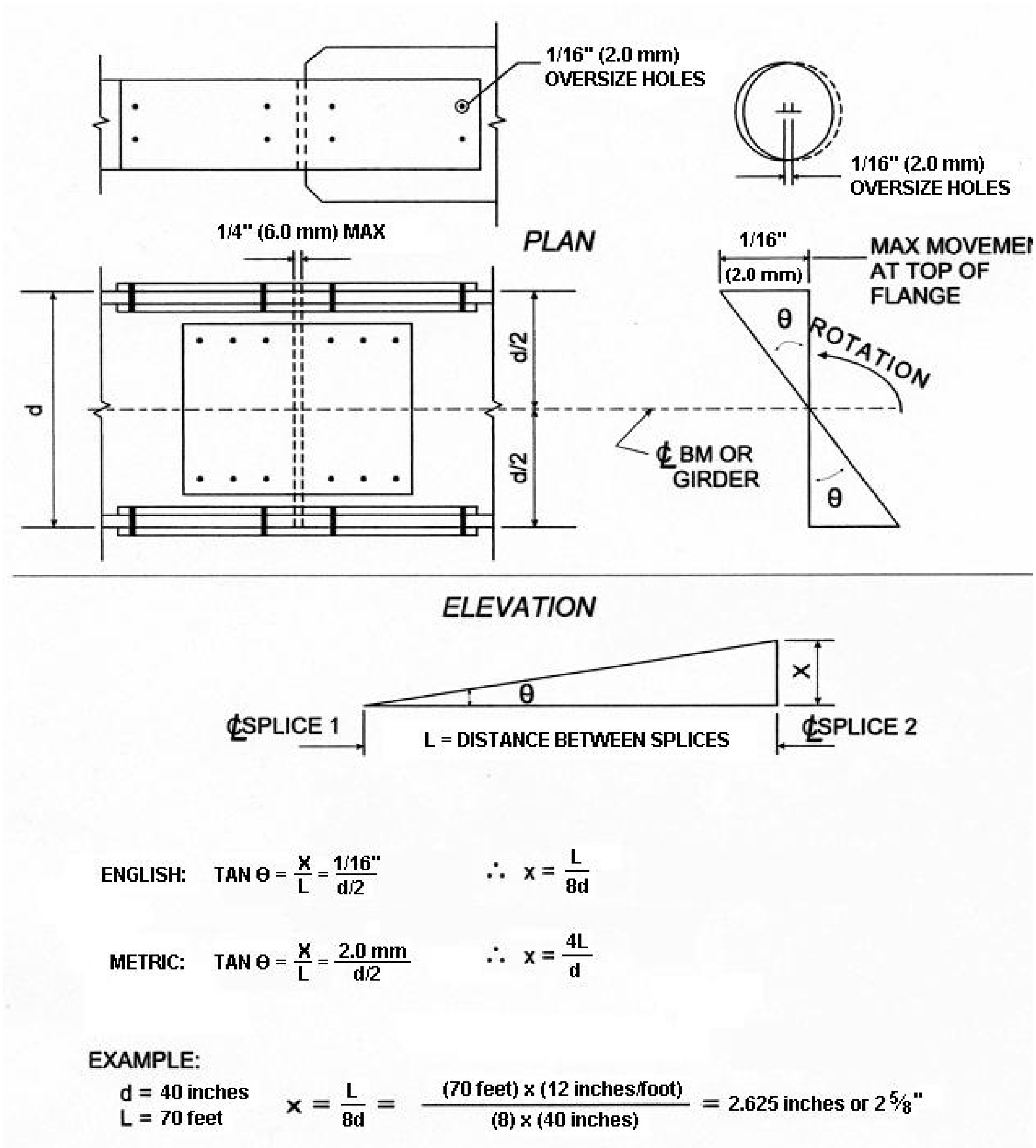


Figure 5-42: Splice Elevation Adjustment Calculations

Next, the bolting crew should start to "snug" the bolts and nuts. "Snug" is defined as the point at which the air wrench begins to impact solidly. A bolt can be snugged-up by a worker using a spud wrench. When the crew completes the "snugging" of the entire connection, the gap between the plates should have entirely disappeared.

Now the drift pins are knocked out, and the remaining holes are filled with bolts and turned to "snug". The connection is now ready for final tensioning.

In Figure 5-43**,** the bolts have been numbered to show the suggested tightening sequence. Bolts and nuts should always be tightened progressively away from the center, the fixed or the rigid points to the free edges.

Final tightening with a torque-control wrench that has been properly calibrated can proceed in a straight-forward manner. For the turn-of-the-nut method, a hand-wrench is used to hold the end not being torqued to ensure that the true required turn measurement is not lost.

Close examination of a nut after final tensioning will disclose slight burrs or peening marks near the edge of each nut "flat". These marks are caused by the "hammering" action of the wrench as it impacts. If nuts on ASTM A325M bolts show no such markings, a thorough inspection should be made to insure that the bolts were properly tensioned. Nuts furnished with ASTM A490M bolts may not show any distortion because of their greater hardness. However, a slight burnishing of the edges should be evident.

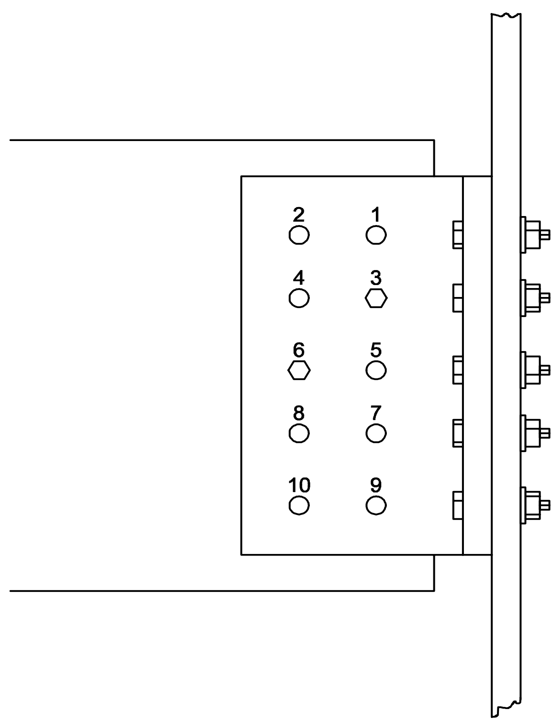


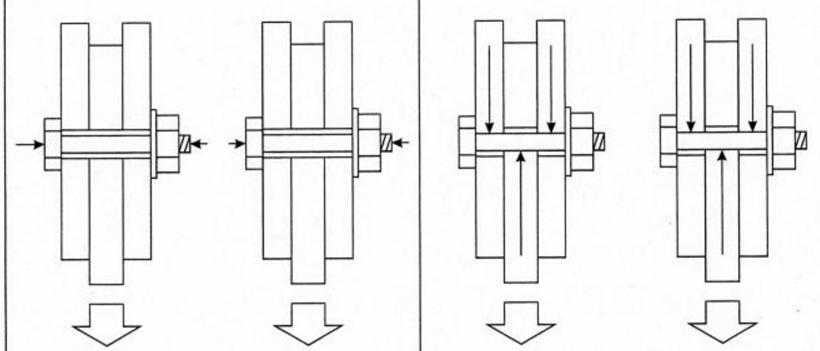
Figure 5-43: Suggested Bolt Tightening Sequence

4. Characteristics of High-Strength Bolts from Tests:

    - Tensile Strength. Mechanical properties for ASTM A325M and ASTM A490M high-strength bolts are determined by applying axial tension loading, either to a full-size bolt or to a reduced section machined from a bolt shank. In normal use, however, the tightening operation induces torsional stress as well as internal tension, and laboratory tests have indicated that the bolts' direct tensile strength is reduced by as much as 5 to 30 percent when tension is induced by torquing. However, it has been proven that high-strength bolts torqued to specifications below failure retain initial ultimate tensile strength once connected.
    - Shear Strength: Research studies have also been undertaken to determine how internal bolt tension affects the ultimate shear strength of the bolt. High-strength bolts were torqued to "snug", ½ turn from snug, and 1½ turns from "snug" and then loaded in direct shear. The initial preload has little influence on shear strength. Bolts which were severely torqued (1½ turns) had approximately the same shear strength as those with little pre-load ("snugged") or nearly maximum pre-load (½ turn).
    - Bolt Clamping Force: The clamping force induced in a high-strength bolt by tightening can be measured by bolt elongation. A calibration curve is used to convert measured elongations to bolt tension or clamping force. Tests have proven that all bolts develop clamping forces in excess of the specified minimum value, and clamping force values are consistent even when elongations vary over a wide range.
    - Moment Connections: Behavior of high-strength bolts in direct tension or combined tension and shear is particularly important in moment connections. End-plate connections are a typical example. When high-strength bolts are pre-loaded in moment connections, the applied moments normally cause little or no increase in bolt tension, especially if the initial pre-load is well above proof load. Since external tensile loads merely change the contact pressure, the bolt does not elongate and cause additional bolt tension. However, additional bolt tension may result from prying action due to distortion of the connection details, depending on the relative stiffness of the connected material and the spacing of the fasteners.

Types of High-Strength Bolt Connections: Your attention is directed to Figure 5-44**5-44.** New Hampshire usually requires washers because when washers are eliminated it is difficult to inspect with torque wrenches.

Figure 31: High-Strength Bolt Connections

CONNECTION	FRICTION-TYPE		BEARING-TYPE	
JOINT PLACEMENT REQUIREMENTS				
BOLT	A 325 (A 325M)	A 490 (A 490M)	A 325 (A 325M)	A 490 (A 490M)
NUT	A 563 (A 563M)	A 563 (A 563M)	A 563 (A 563M)	A 563 (A 563M)
WASHER - ASTM F 436 (F 436M) (TURN-OF-NUT METHOD) OR (CALIBRATED WRENCH METHOD)	ONE	[1] ONE (UNDER TURNED ELEMENT IF Y. P. IS 40,000 PSI (276 MPa) OR GREATER) TWO (IF Y. P. IS LESS THAN 40,000 PSI (276 MPa))	ONE	[1] ONE (UNDER TURNED ELEMENT IF Y. P. IS 40,000 PSI (276 MPa) OR GREATER) TWO (IF Y. P. IS LESS THAN 40,000 PSI (276 MPa))
THREAD LOCATION	MAY BE IN SHEAR PLANE	MAY BE IN SHEAR PLANE	MUST BE OUT OF SHEAR PLANE WHEN HIGHER ALLOWABLE VALUE FOR BEARING-TYPE CONNECTION IS USED.	MUST BE OUT OF SHEAR PLANE WHEN HIGHER ALLOWABLE VALUE FOR BEARING-TYPE CONNECTION IS USED.

[1] YIELD POINT (Y. P.) REFERS TO THAT OF THE MATERIAL BEING JOINED.

Figure 5-44: High-Strength Bolt Connections

F. Welding:

If the contractor requests to field weld a connection not shown as a field weld on the plans, then the Bridge Design Engineer and/or District Construction Engineer must give approval for each field weld requested. Field welding shall meet the requirements of Standard Specification 550.3.16. Welders shall have passed AASHTO welding specification qualification tests. The minimum qualifications shall be for “all-position” groove welding on “limited thickness (3/4 in [20 mm] maximum) plate. The welder must furnish acceptable proof of these qualifications, namely AWS Certification, sworn copies of a satisfactory qualification test record, or both.

1. <u>Welding Inspection and Troubleshooting:</u> Use the troubleshooting chart on the next four pages as an aid for welding inspection.

Figure 5-45: Welding Troubleshooting Chart

TROUBLE	CAUSE	CURE
SHALLOW PENETRATION	Welding speed too fast.	Use speeds recommended in procedure tables.
	Electrode too large.	Use small electrodes for deeper grooves.
	Current too low.	Use adequate current for deep penetration.
	Faulty preparation.	Allow some gap (free space) at bottom of joint.
DISTORTION OR WARPING	Uneven heating of part.	Preheat before welding. Post heat after welding. Make small welds at several points on part to distribute heat evenly. Weld in solid bars or rounds where much buildup is required. Back step, welding intermittently. Use fewer passes where possible.
	Welding wrong area.	Perform all forming operations prior to welding. Do not over-weld. Assure proper edge preparation and fit-up.
	Shrinkage of weld bead.	Tack or clamp parts properly in rigid fixtures. Preform parts to counteract distortion. Peen bead on cooling.

Figure 5-45: Welding Troubleshooting Chart (Continued)

TROUBLE	CAUSE	CURE
CRACKING	Base metal not within recommended analysis range.	Use a less penetrating electrode or decrease current. Change steel analysis. Shorten arc length to make beads more convex. Decrease welding current and use more passes. Decrease welding speed. Use low-hydrogen type electrode, such as the Jetweld LH-70. Back step to penetrate previous beads, reducing the concentration of undesirable elements in the weld crater. Preheat parts before welding.
	Improper joint preparation.	Leave 1/32” (0.8 mm) gap between plates for free movement. Position and weld slightly uphill (about 5 degrees) to increase weld section on first pass.
	Weld joint is too rigid.	Weld towards unrestrained end of weld joints.
	Welds too small or wrong shape.	Change to less penetrating type of electrode. Decrease weld current, using more passes.
UNDERCUTTING	Welding current too high.	Correct current and travel speed. Decrease electrode size.
	Faulty electrode manipulation for job.	Weld slowly, using lower current. Avoid excessive weaving; however, use uniform weave in butt welding.

Figure 5-45: Welding Troubleshooting Chart (Continued)

TROUBLE	CAUSE	CURE
SURFACE HOLES & POROSITY	Low carbon in base metal.	Be certain that carbon and manganese content are sufficiently high; sulfur, phosphorous and silicon sufficiently low.
	Poor electrode selection.	Change to a low-hydrogen type electrode, such as the Jetweld LH-70.
	Insufficient puddling.	Allow sufficient puddling time for gases to escape.
	Excessive arcing.	Decrease current, using a shorter arc.
	Steel is outside recommended analysis range.	See “CRACKING”.
ARC BLOW	Stray magnetic fields cause arc to deviate from its intended course when welding with DC.	Change to AC if at all practical. This is by far the best remedy. Shift ground clamp to other end of work. Hold as short an arc as possible. Change to a smaller electrode. Lower welding current. Weld to a heavy tack, or weld already made. Weld in same direction as arc blow. Use steel blocks to alter current path around arc. Change to alternate electrode with greater arc force. Use back stepping on long welds. Tack a small plate across seam at weld end. Use run-out tabs. Ground the work at several spots.

Figure 5-45: Welding Troubleshooting Chart (Continued)

TROUBLE	CAUSE	CURE
WELD SPATTER	Current too high.	Check current settings for electrode size and thickness used.
(Although weld spatter does not affect structural strength, excessive spatter has a poor appearance and will increase cleaning cost)	Wrong electrode choice.	Use an electrode with minimum-spatter characteristics.
	Wrong polarity.	Employ correct polarity for electrode.
	Too large an electrode.	Use correct electrode diameter.
	Wrong electrode angle.	Use electrode angle recommended in procedures tables.
	Arc blow.	Apply corrections for arc blow. (Make certain electrodes are perfectly dry).
POOR FUSION	Current setting too low.	Be certain that proper current is being used. Use stringer bead technique.
	Dirt and ragged edges in joint.	Be certain that weld is clean – sides of joint are smooth.
	Welding electrodes too large for groove or fillet.	Use smaller electrodes or alternate electrode. Change joint preparation.
ROUGH WELDING	Moisture pick-up.	Store electrodes in cabinet or room about 10 degrees warmer than the surrounding atmosphere. If electrodes have become damp, dry at 200^0F (90 °C) for one hour. Low hydrogen electrodes require $300\text{-}700^0$ F (150 - 370 °C) drying temperatures.

2. Welding Symbols:

Figure 5-46 shows welding symbols based on the old British Standard, BS499. They are very similar, if not identical, to the American Welding Society standard. This information may also be found online at: http://www.gowelding.com/weld/symbol/symbol.htm#_AWS.

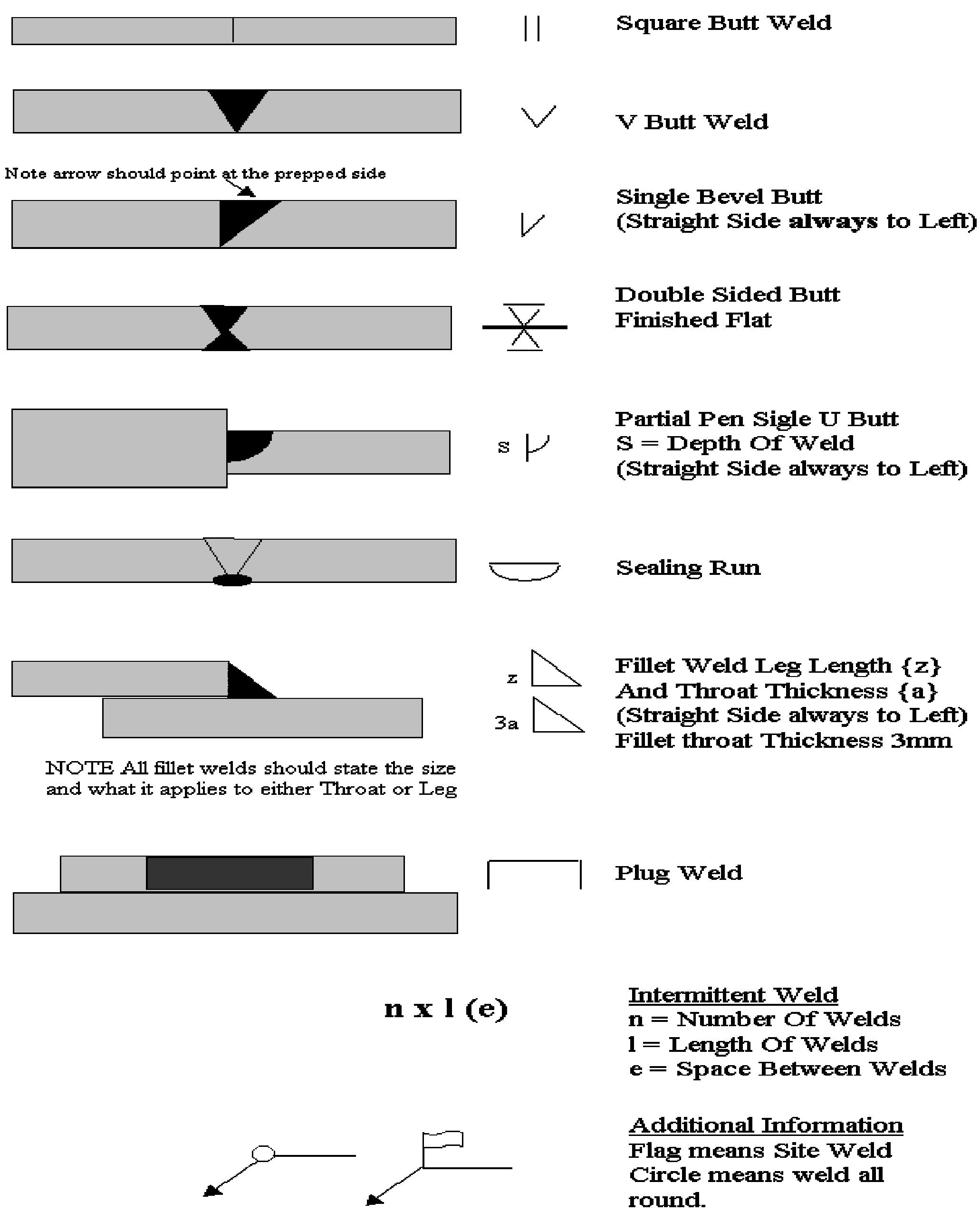


Figure 5-46: Sample Welding Symbols

For American Welding Society (AWS) welding symbols see Figure 5-47 or go to website: http://www.aws.org/technical/errata/A2.4errata.pdf

Errata for AWS A2.4-98, Standard Symbols for Welding, Brazing, and Nondestructive Examination.
The following is the corrected Welding Symbol Chart for AWS A2.4-98, pages 106 and 107.

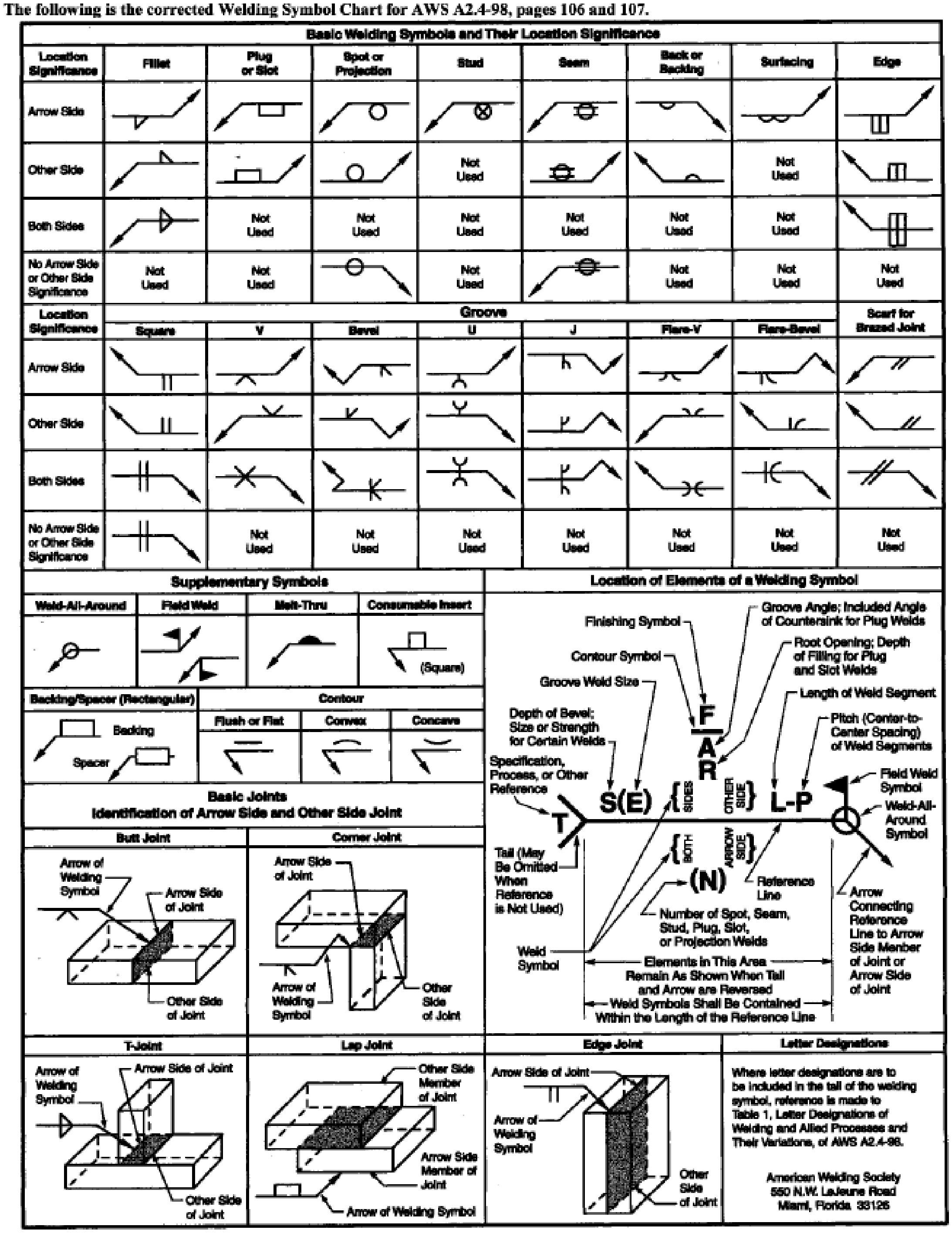
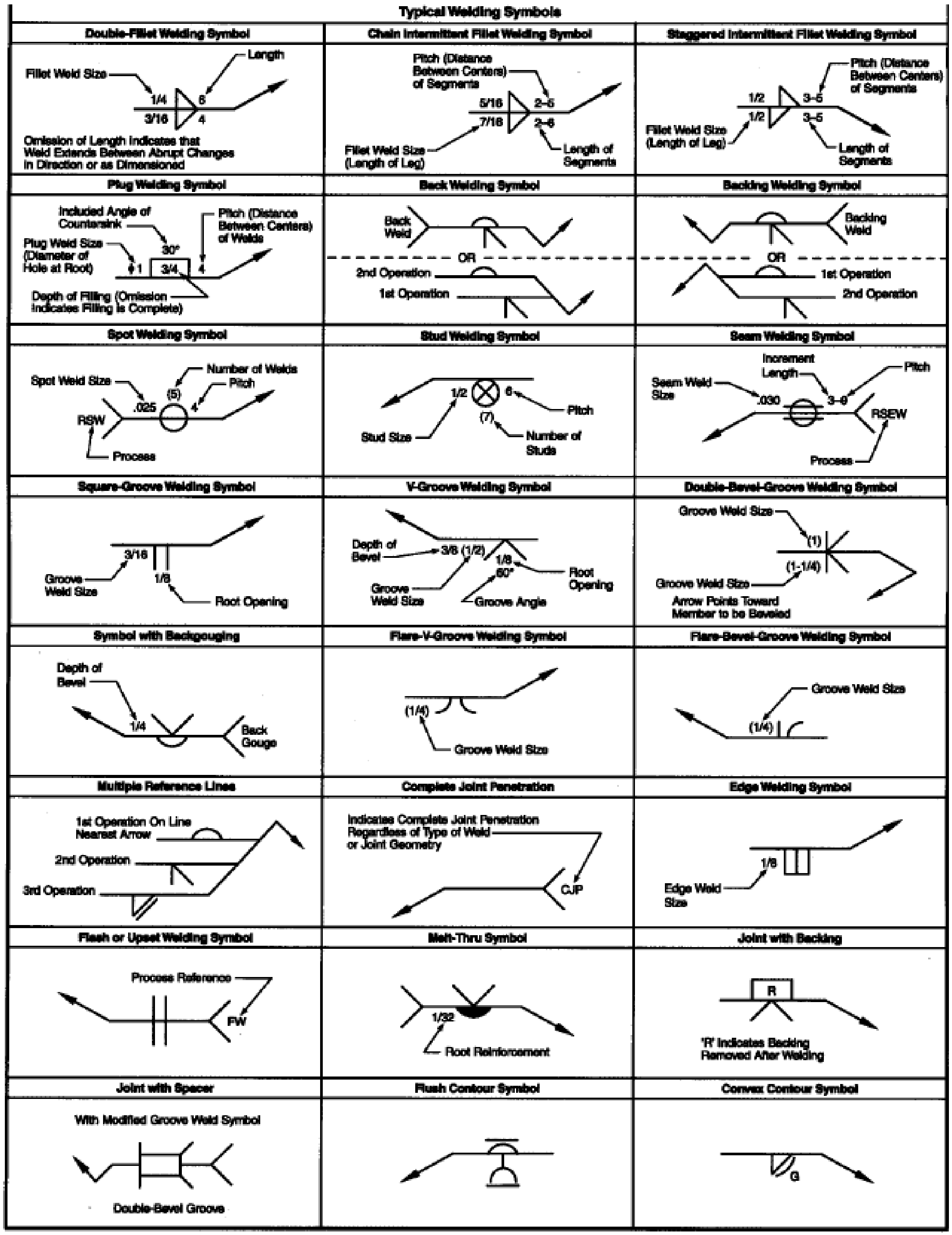


Figure 5-47: AWS Standard Welding Symbols



It should be understood that these charts are intended only as shop aids. The only complete and official presentation of the standard welding symbols is in AWS A2.4-98, Standard Symbols for Welding, Brazing, and Nondestructive Examination.

Figure 5-47: AWS Standard Welding Symbols (Continued)

G. Camber and Haunch Grades:

Vertical alignment of a bridge deck is defined by a profile grade line in the same manner that a roadway grade is defined. A bridge deck generally consists of a concrete slab that must be built correctly to grade on the first try. So, all girders should be checked for camber as soon as they are erected, with diaphragms and splices completed, by shooting elevations at beam locations shown on the plan deflection diagram (generally the 10th points between piers and/or abutments.. If a girder shows an excessive amount of camber (plus or minus) that cannot be attributed to and corrected by erection splicing (see method of adjusting splices previously discussed under bolt tensioning), adjustments can be computed into the haunch grades. Where the computed haunch grades would indicate less than a zero haunch, a haunch varying much from the typical shown, or a haunch so high that the shear connectors are not in the lower mat of reinforcing steel, the Bridge Design Engineer and the District Construction Engineer should be consulted.

If the above goes unattended on single span bridges, a “hump” in the bridge rail may be very distinct, particularly if the bridge has wings parallel to the deck and the same line of rail runs from the deck through the wings.

H. Sandblasting:

Prior to painting, all steel must be sandblasted clean either by the brush off, commercial, or near white method. These methods are determined by the degree of deterioration or corrosion. The contractor will be required to furnish a booklet from S.E.P.C. with photos of various degrees of sandblasting. This will be provided through the contract.

If old lead based paint is being removed, it is very important to hold preliminary meetings with the contractor to discuss safety, access, containment, toxic disposal etc.

I. Painting:

The Contractor should submit a 5 gal. (18.9 L) can of each type of paint to the Bureau of Materials and Research prior to painting. Where more than one lot number is to be used, each lot is to be submitted for testing. As previously mentioned, paint found on contact surfaces should be sandblasted prior to erection. Surfaces that will be inaccessible after erection will have to be painted with the full number of coats prior to erection. The CA should ensure that the epoxy paint systems ordered on the plans are applied.

SECTION 562 - ELASTOMERIC SEALANT

562.1 - GENERAL

Elastomeric sealant is a synthetic rubber compound with elastic properties designed to give a tightly sealed joint, and is most generally used in expansion joints, such as between bridge wingwalls and abutments.

562.1 - MATERIALS

The Standard Specifications adequately describe material to be used for the type of joint being sealed. Unless otherwise specified, the color of the mixed sealing compound should be that which blends with adjacent surfaces. For joints that move in a shearing direction (i.e. deck to pilaster) specific materials shall be used as directed by the Bureau of Materials and Research.

All sealants used must be on the Qualified Products List. A Certificate of Compliance from the manufacturer must also be obtained, and labeling on cans should be examined for correct contents before use.

562.3 - CONSTRUCTION OPERATIONS

After proper application of the sealant, as described in the Standard Specifications, the sealing compound should maintain the properties of a tack-free, cured, rubber-like material that protects the joint against the infiltration of water and other foreign matter. Application of all elastomeric sealants should be completed prior to application of other concrete sealers and waterproofers.

SECTION 563 - BRIDGE RAILING

563.1 - GENERAL

Bridge railing adds safety and beauty to the bridge. It requires the detailed attention of the contract administrator in the layout of rail anchor bolts and the alignment of the erected rail.

563.3 - CONSTRUCTION OPERATIONS

Anchor bolt spacing as shown on the plans should be checked carefully before any layout work is done. Since sidewalk concrete is often placed in intermediate sections, an error in one section will affect the spacing across the entire structure. Anchor bolt layout for the entire structure should be field checked before permanently setting any one section.

Anchor bolt units should be set carefully with the aid of a template and secured firmly to prevent movement during concrete placement. Welding extra resteel to the anchor bolts to hold them in place is not acceptable. It is critical that the resteel and anchor bolt assemblies be installed per plan (i.e. no cutting of or welding on the coping steel). Any cutting or welding that is not approved may weaken the rail and coping system so that it no longer withstands crash testing requirements. In recent years, bridge copings have been widening to allow easier installation of the anchor bolts assemblies within the coping resteel.

The exposed length of anchor bolts above the sidewalk concrete should be computed by adding the dimensions of the preformed bearing pad, post base, washer, nut, and exposed thread, and checked against the plans and shop drawings. As a good practice, the contractor should tape or cover the exposed threads prior to placing the sidewalk concrete to minimize the amount of cleaning necessary to install the nuts. When the posts are erected, the bearing surfaces should be in full contact. Be sure that sufficient shims have been allotted to align rail posts as noted on the plans, either plumb or normal to grade. Insure that all hardware on steel or aluminum rail is affixed properly. Final adjustments of rail should present a pleasing line and grade.

On bridges where the rail is continuous through wings and span, a check should be made of the rail location as soon as the deck is poured to see if the expected dead load deflection has been substantially realized. If it has not, some adjustment will be necessary to carry the excess camber through the full length of the rail; otherwise, a hump in the span portion will be very obvious. For a relatively small amount of correction where no part of the wings have already been poured to grade, adjust the curb and sidewalk elevations to distribute the excess height at the center of the span through the full length of rail making the hump long, smooth and considerably less objectionable. If the curb and sidewalk cannot be sufficiently adjusted, the second solution is to raise some of the posts to provide a pleasing appearance to the entire rail. This can be done by raising some of the anchor bolts and by either shimming or grouting under the posts. A contract administrator with this problem should discuss these possible solutions and the specific details with their District Construction Engineer.

SECTION 564 - BRIDGE LIGHTING SYSTEM

564.1 - GENERAL

The bridge lighting system includes anchor bolts, junction boxes, fittings, and conduits that are a permanent part of the bridge structure. Proper installation of the system will allow the local electric utility to erect light poles and wire lights after the bridge is completed.

564.2 - MATERIALS

All materials used under this specification must be covered by Certificates of Compliance.

564.3 - CONSTRUCTION OPERATIONS

A. Coordination with Utility:

Before the contractor commences any work associated with the lighting system, it is a good practice to check with the supplying electric utility to determine whether sizes of conduits, junction boxes, design of fittings, and the number, sizes, and locations of anchor bolts are correct. This has usually been done in the design stage but experience has shown that the utility sometimes has a change of policy or equipment between the time of design and actual construction that will affect the system.

B. Anchor Bolts:

The contract administrator should review the plans to see whether the anchor bolts for the light standards are incorporated in the deck concrete pour or just the sidewalk/curb pour. Also, the location and thread reveal should be checked just before pouring concrete and just after finishing.

C. Bearing Surface:

If aluminum light pole standards are to be used on concrete bridges, the contact surface of the base should be painted with asphaltic paint or separated from the concrete by a suitable pad.

D. Conduit and Junction Boxes:

The conduit system should be completed and satisfactorily anchored before concrete is poured. At the ends of the bridge, be sure that conduit is placed before the approach slabs are poured and extends well beyond the slab so it can be connected to later.

SECTION 566 - ELASTOMERIC BRIDGE JOINT SEAL

566.1 - GENERAL

This item is normally used in expansion dams, in sidewalks, and between parallel bridge decks where a water-tight joint is needed.

566.3 - CONSTRUCTION OPERATIONS

In order to obtain a good joint, the end dam steel should be cleaned of dirt, oil and concrete spatterings before installation of the neoprene. The elastomeric joint seal itself should be clean of dirt, oil and dust so that the adhesive lubricant bonds properly to it. On some plans, notes have been included to field clean the seal with solvent immediately before use. Check the plans and the manufacturer's instructions completely to insure that all proper action is taken during installation. If sufficient lubricant adhesive material is used, the seal should be easier to install and a water-tight joint should be obtained. Care should be used to provide a smooth S-curve where the roadway and sidewalk meet and to get the proper depth across the sidewalk. Special attention should be paid to notes on the plans regarding relief cutting of the seal where this S-curve is made. Improper cutting can cause structural damage to the seal and seriously impair its effectiveness. In general, the installation of the joint seal should be one of the last operations in the construction of the bridge, preferably after the roadway has been paved. Trucking over the seal prior to paving can result in stone being forced into the seal and cutting it.

SECTION 568 - STRUCTURAL TIMBER

568.1 - GENERAL

This item is seldom used and is well covered in the Standard Specifications.

SECTION 570 - STONE MASONRY

570.1 - GENERAL

This item is well-explained in the Standard Specifications. However, the specifications contain some terminology that may be unfamiliar to the some inspectors. The following glossary defines some of these terms:

Stretcher - A stone with its long axis laid parallel to the face of the wall.

Header - A stone with its end toward the face of the wall.

Lewis - An iron dovetailed tenon, made in sections, which can be fitted into a dovetail mortise to hoist stones.

Cramp - A device, usually of iron bent at the ends, used to anchor mortared blocks of stone.

Voussoir - Any of the wedge-shaped pieces of which an arch or vault is composed.

Centering - Falsework for a masonry arch.

Intrados - The interior curve of an arch.

Extrados - The exterior curve of an arch.

One problem that often occurs with stone masonry is efflorescence. For a discussion of efflorescence and recommended ways to remove it, see Section 520.3, subsection "T. Efflorescence" in this manual.

SECTION 582 - SLOPE PAVING WITH CONCRETE

582.1 - GENERAL

Slope paving is primarily constructed at structures, and at times in other locations. Its purpose is to give an aesthetic appearance, as well as to stabilize slopes that cannot otherwise support plant growth for stabilization.

582.2 - MATERIALS

Description of materials to be used for slope paving is adequately given in the Standard Specifications.

582.3 - CONSTRUCTION OPERATIONS

Although the Standard Specifications give a good description of what the finished product should look like, the plans should be checked to determine whether or not additional detail is required. Extra depths of bedding or a gravel course below the bedding may be called for in some instances.

Fine grading of the bedding is a must to provide a well-aligned and uniform appearing product. In most cases, the bedding can be formed, graded and screeded in preparation for laying the slope paving.

SECTION 583 - RIPRAP

583.1 - GENERAL

Riprap protects embankments, dikes, and channels from eroding under the effects of relatively low-velocity water. Riprap provides relatively smooth, tight protection against wash and scour, whereas stone fill is large and rough, intended to dissipate the energy of high-velocity turbulence.

583.2 - MATERIALS

In addition to requirements stated in the Standard Specifications, the contract administrator should encourage uniformity in the type of stone used and the finished appearance. Therefore, if the contractor begins with quarry stone, there should be enough to complete the entire installation. In addition, if a gravel blanket is not shown on the plans, the contract administrator should check to insure that the subgrade material is satisfactory to support the stone and is not of a gradation that will be eroded by water circulating through the riprap.

583.3 - CONSTRUCTION OPERATIONS

A. Field Check:

The contract administrator should thoroughly review the plans for locating the riprap and note any field conditions that require adjustments in the preliminary design. The riprap installation should be located in proper respect to high water elevation; direction of flow and angle of impingement; type and security of trees and vegetation; and any springs or drainage water courses that might affect the stability of the design. The installation should also be blended into already stable areas.

B. Layout:

The contract administrator should approve the contractor's layout before stone work is begun and should verify not only the plan location, but confirm subgrade and riprap elevations. The Contractor must exercise care not to disturb old ground adjacent to the riprap installation.

C. Placing Stone:

Riprap done properly should be carefully supervised, with workers choosing the best face of the stones to expose and setting each stone to grade. An effort should be made to obtain and use clean stone. The subgrade must be even and well-compacted before placing any stone.

D. Inspection:

The inspector should check the size of the stone to insure conformity with the specifications. The individual stones should be placed in contact with each other. All open joints shall be filled with smaller stones firmly rammed into place.

E. Policy:

Riprap construction should be coordinated in order to be accomplished when the embankment or channel subgrade is completed and before erosion is allowed to take place.

SECTION 585 - STONE FILL

585.1 - GENERAL

Stone fill will consist of Class A, B, C or D stone as specified on the plans and described in the Standard Specifications. The purpose of stone fill is to prevent erosion. The area designated to receive stone fill should be studied to see that the plan design is proper and adequate to meet field conditions. The contract administrator should recommend to the District Construction Engineer any extension or modification of the stone fill if there is doubt concerning the adequacy of the plan design.

585.2 - MATERIALS

Stone sizes should be checked to be sure they conform to requirements for the class specified.

585.3 - CONSTRUCTION OPERATIONS

The construction of stone fill should be accomplished in a timely manner when access is most available. For example, stone fill should be placed and graded in the toe of embankments before the embankment is built to an appreciable height. Also, stone fills in front of abutments should be constructed prior to steel erection.

Batter boards or string lines erected on tall 2 x 4 lumber stock and marked, for example, “cut 2 ft to subgrade” or “cut 1 ft to top of stone fill,” are usually necessary to obtain proper grading. The contractor should be encouraged to set these types of batter boards.

When a seal of stone spalls is specified or ordered it should consist of uniformly graded rock fragments ranging downward from 1 ft^3 (0.03 m^3) in size. The completed blanket should be made as void-free as possible by working and compacting the spalls.

After stone fill has been completed, the contract administrator or inspector should check surface stones to insure that they are chinked and securely locked in place, giving a pleasing finished appearance. If stone fill is placed along a stream likely to be fished, be sure that there are no stones that can be easily dislodged.

SECTION 591 - STRUCTURAL PLATE PIPES, PIPE-ARCHES, AND ARCHES

591.1 - GENERAL

This item is most often used when an economical structure is needed for short spans, as in small watercourses or passageways such as a cattle pass.

591.2 - MATERIALS

Materials must comply with the Standard Specifications, and Certificates of Compliance must accompany delivery.

591.3 - CONSTRUCTION OPERATIONS

The excavation and bedding requirements are well covered in the Standard Specifications other than in the case of watercourse structures. Be sure that the granular backfill material used is gravel, both in bedding and backfill. Sand should not be used in this case, as sand becomes super-saturated if bed locations are not kept completely dry, and proper compaction cannot be obtained. Also, if flash floods or very high and fast water conditions occur in a stream, sand may be more subject to undermining and washout.

In most cases, structural plate pipe and pipe arches are preassembled or need only to be bolted in sections. When a structure is to be assembled in place, detailed erection instructions will be shipped. In this case, it is necessary to follow the erection and bolting procedures of the manufacturer and the Standard Specifications.

Usually when a structure is shipped preassembled, the galvanizing is done at the manufacturer's plant, and the contract administrator need only check for any bruised or broken places in the spelter coating.

Backfilling is well-covered in the Standard Specifications but it should be stressed that thorough compaction is needed under the haunches of a pipe or pipe-arch, and should be done equally on each side. The strength of the structure, as well as the provision of a stable roadway without future settlement, is dependent upon a well-compacted backfill. It is imperative that a properly formed and compacted bed be used; otherwise, there may be considerable difficulty in bolting up the sections.

SECTION 593 – GEOTEXTILES

593.1 - GENERAL

Refer to the Qualified Products List to determine the type of geotextile required based on its application, strength class, structure, and filter category.

A supplemental specification was written in January of 2004 to help clarify when and what type of geotextile should be used. In summary, geotextile "applications" are described based on its most common use; "strength classes" range from low to extra high strength and are defined by AASHTO M 288 or ASTM standards; "structure" defines the basic composition of the fabric; and "filter category" specifies the required permittivity and apparent opening size (AOS) properties of the fabric. The specifications and Qualified Products List should be used to make sure the proper fabric is being used for the required purpose.
The Qualified Products List may found at the following website: http://www.nh.gov/dot/materialsandresearch/pdf/apl.pdf.